Seismic analysis of concrete structures within nuclear industry

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Abstract

Earthquake has always been a hazard for civil structures and keeping the structures integrity during and after an earthquake is of vital importance. This phenomenon’s impact is sudden and there is little or no warning to make the preparations for this natural disaster. Much damage has been done on structures which have led to major collapses and loss of many lives. Civil structures such as nuclear power plants are designed to withstand earthquakes and in the event of a major seismic event, to shut down safely.

The aim of this thesis is to present the seismic design procedures for concrete structures, in basic and detailed design, according to Eurocode 8. Also to describe and understand the difference between Eurocode 8 and the DNB in seismic analysis of nuclear power plants. To evaluate the use of DNB instead of Eurocode 8 with Swedish seismic conditions is also another aim in this thesis.

Loads and actions which apply on a structure in a seismic design and corresponding load combinations are presented for Eurocode 8 and the DNB. An example is also given to clarify the design of primary seismic beams and columns with high ductility class (DCH). A case study of a nuclear structure from a test project named SMART2013 has been made by analyzing and comparing the results from Eurocode 8 and the DNB with a finite element model in FEM-Design software. Natural frequencies of the model are compared with the tested model in SMART2013-project to evaluate the finite element modeling. The model is seismically analyzed with load combinations from Eurocode 8 and the DNB with Swedish elastic ground response spectrum with the probability of $10^{-5}$. Results obtained from the primary seismic beams and columns are compared and analyzed.

Being on the safe and conservative side of the design values is always preferred in seismic analysis of a vital and sensitive structure such as nuclear power plants. The results from this thesis shows that, purely structural, combination of Swedish elastic ground response spectrum with the Eurocode 8 load combination will give more conservative values than the DNB.

Key words: Earthquake, nuclear power plants, seismic design, Eurocode 8, DNB, load combination, primary seismic beam, primary seismic column, high ductility class (DCH), SMART2013, finite element analysis, natural frequency, seismic analysis, elastic response spectrum, ground response spectrum
Sammanfattning

I stora delar av världen har jordbävningar alltid varit ett hot för byggnaders integritet. Karaktären av en jordbävning är plötslig och föranleds av små eller inga varningar. Om jordbävningen medför att byggnader kollapsar sker ofta stora förluster av människoliv direkt eller indirekt. Kärnkraftsverk är anläggningar som dimensioneras för att klara jordbävningar och ska kunna gå till säker avställning vid en sådan händelse.

Syftet med föreliggande rapport är att presentera hur betongkonstruktioner dimensioneras för jordbävning enligt Eurokod 8. Rapporten redogör även för skillnader mellan att dimensionera enligt Eurokod 8 och DNB (Dimensionering av nukleära byggnadskonstruktioner) samt hur det slår att använda Eurokod med svenska seismiska förhållanden.

Laster och lastkombinationer som används vid jordbävningsdimensionering av betongbyggnader är presenterad enligt både Eurokod och DNB. Ett exempel presenteras för att visa hur primära balkar och pelare med hög duktilitetsklass (DCH) dimensioneras för seismisk påverkan. En fallstudie av en nukleär byggnad från ett internationellt projekt, SMART2013, har använts för att analysera och utvärdera resultaten från Eurokod och DNB. Byggnaden har analyserats med finita element med programvaran FEM Design. Modellens riktighet har verifiserats genom att jämföra bland annat egenfrekvenser med de från officiella rapporter från SMART2013. Byggnaden är analyserad för seismisk last enligt svenska förhållanden med markresponsspektra $10^{-5}$, och primära balkar och pelare har analyserats och utvärderats enligt både Eurokod och DNB.

Nyckelord: Jordbävning, kärnkraftverk, jordbävningsdimensionering, Eurokod 8, DNB, lastkombination, primära seismiska balkar, primära seismiska pelare, hög duktilitetsklass (DCH), SMART2013, finita element analys, egenfrekvens, seismisk analys, elastiskresponsspektra, markresponsspektra.
List of notations

$A_{tw}$ The cross sectional area of one stirrup

$A_{Ek}$ Characteristic seismic action

$A_{Ed}$ Designed seismic action

$A_c$ The concrete area of the cross-section

$c_t$ The thermal coefficient

$c_e$ The exposure coefficient

$D_o$ The diameter of confined core (to the centerline of hoops)

$D_k$ Dead weight

$D$ Displacement

$E_{Ej}$ The value of the considered seismic action effect on the vibration mode $j$

$E_{EI}$ The value of the considered seismic action effect on the vibration mode $i$

$E_E$ The seismic action effect under consideration

$E_{DBE}$ Seismic load due to designed DBE

$F_i$ The horizontal force acting on storey $i$

$G_{k,j}$ Characteristic value of permanent action

$H_{qw}$ Water pressure difference between normal water level and time variable water level

$H_{qe}$ Soil pressure due to movable surface load

$H_{gw}$ Water pressure

$H_{ge}$ Earth pressure

$L$ Live load

$L_i$ The floor dimension perpendicular to the direction of the seismic action

$\sum M_{Rc}$ The sum of design values of the moments of resistance of the columns framing the joint

$\sum M_{Rb}$ The sum of design values of the moments of resistance of the beams framing the joint

$M_{ai}$ The torsional moment applied at storey $i$ about vertical axis

$M_n$ Process related loads during normal operation and shutdown period

$M_d$ Process related loads during operation disturbance

$N_{Ed}$ The normal force from tensioning or external pressure
Pre-stressed force

Characteristic value of the accompanying variable action \( i \)

Characteristic imposed point load

The design spectrum

Snow load

Period of vibration

The period of vibration of mode \( k \)

The vibration periods of mode \( j \)

The vibration periods of mode \( i \)

Upper limit of the period of the constant spectral acceleration branch

Lower limit of the period of the constant spectral acceleration branch

Value defining the beginning of the constant displacement response range of the spectrum

Natural period of vibration

The fundamental period of the building within vertical plane

Shear resistance of each structural wall \( i \)

The force that is needed to be taken by shear reinforcement

Shear load capacity

Web compression failure

Velocity

Wind load

The storey height in meters

Natural frequency of vibration

Height of the primary seismic beam

Height of the wall \( i \)

The largest cross-section of the column

Depth of confined core (to the centerline of hoops)

The design ground acceleration
\( b_c \)  
Gross cross-sectional width

\( b_c \)  
The largest cross-sectional dimension of the column normal to the beam axis

\( b_o \)  
Minimum dimension of the concrete core (to the inside of the hoops)

\( b_o \)  
Width of confined core (to the centerline of the hoops)

\( b_i \)  
Distance between consecutive engaged bars

\( b_w \)  
Width of the primary seismic beam

\( b_{wo} \)  
Thickness of the web

\( d_{bL} \)  
Minimum dimension of the transverse bars

\( d_{bw} \)  
Minimum diameter of shear reinforcement

\( e_0 \)  
Structural eccentricity

\( e_{0x} \)  
The distance between the center of stiffness and the center of mass, measured along \( x \)-direction, which is normal to the direction of analysis considered

\( e_{ai} \)  
Accidental eccentricity of storey mass \( i \)

\( f_0 \)  
Peak value of the earthquake induced resisting force

\( f_{ck} \)  
Characteristic compressive strength of concrete

\( f_{ctd} \)  
The design value of concrete tensile strength

\( f_{ywd} \)  
Yield strength of the shear reinforcement

\( f_y \)  
Yielding stress

\( k_w \)  
Factor reflecting the prevailing failure mode in structural systems with wall

\( l_{ot} \)  
Distance between torsional restraints

\( l_{b,\text{min}} \)  
Minimum anchorage length

\( l_{cr} \)  
Length of the critical region from the connecting joint

\( l'_{cr} \)  
Critical region of the first two storeys

\( l_{wi} \)  
Length of the section of wall \( i \)

\( l_{cl} \)  
Clear length of the column

\( l_s \)  
Radius of gyration of the floor mass in plan

\( q_0 \)  
Basic value of the behavior factor

\( r_{ij} \)  
Interaction between two vibration periods taking into account the declining ratio
\( r_x \)  
Square root of the ratio of the torsional stiffness to the lateral stiffness in \( y \)-direction ("torsional radius")

\( s_k \)  
Characteristic value of snow load

\( u_0 \)  
Peak value of the earthquake induced resisting deformation

\( u_m \)  
Maximum deformation

\( u_y \)  
Yield deformation

\( v_d \)  
Normalized design axial force

\( v_{\text{min}} \)  
Minimum shear force capacity of the concrete

\( q_k \)  
Characteristic imposed line load

\( \Delta T \)  
Climate related temperature load

\( x \)  
The concrete compression zone height

\( \nu \)  
The reduction factor

\( s \)  
Distance between stirrups

\( r \)  
Torsional radius

\( q \)  
Behavior factor

\( n \)  
The number of storeys above the foundation/the top of a rigid basement

\( n \)  
The total number of longitudinal bars laterally engaged by hoops or cross ties

\( k \)  
The number of modes taken into account

\( f \)  
Natural cyclic frequency

\( b \)  
The width of compression flange

\( a \)  
Acceleration

\( m_z \)  
Effective mass moments in \( z \) direction

\( m_y \)  
Effective mass moments in \( y \) direction

\( m_x \)  
Effective mass moments in \( x \) direction

\( h \)  
The total depth of beam in central part of the distance between torsional restraints

\( \omega_n \)  
Natural circular frequency

\( \mu_g \)  
The displacement ductility factor

\( \gamma_C \)  
Partial coefficient for concrete strength
\( \alpha_{tu} \) The value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant

\( \alpha_t \) The value by which the horizontal seismic design action is multiplied, in order to first reach the flexural resistance in any member of the structure, while all other design actions remain constant

\( \alpha_5 \) The coefficient for Effect of the pressure transverse to the plane of splitting along design anchorage length

\( \alpha_4 \) The coefficient for Effect of welded transverse bars

\( \alpha_3 \) The coefficient for Effect of confinement by transverse reinforcement

\( \alpha_2 \) The coefficient for Effect of concrete minimum cover

\( \alpha_1 \) The coefficient for Effect of bar form assuming adequate cover

\( \alpha_0 \) Defined as the prevailing aspect ratio of the walls of the structural system

\( \varepsilon_{sy,d} \) The design value of tension steel strain at yield

\( \varepsilon_{cs} \) Shrinkage

\( \delta_s \) Settlement

\( \gamma_{p, unfav} \) Partial factor for pre-stressing actions

\( \omega_{wd} \) The mechanical volumetric ratio of confining hoops within the critical region

\( \psi_{E,i} \) Combination coefficient for variable action \( i \)

\( \psi_2 \) Factor for quasi-permanent value of a variable action

\( \psi_{2,i} \) Factor for quasi-permanent value of a variable action \( i \)

\( \psi_1 \) Factor for combination of frequent values of a variable action

\( \psi_0 \) Factor for combination value of a variable action

\( \sigma_{sd} \) The stress corresponding to the design value

\( \sigma_{cp} \) The average compressive stress

\( \rho_1 \) Reinforcement content

\( \mu_\theta \) Curvature ductility

\( \mu_p \) Required value of the curvature ductility factor

\( \mu_i \) Snow load shape coefficient

\( \eta_2 \) The coefficient related to the bar diameter
\( \eta_k \)  
\( \phi_y \)  
\( \phi_u \)  
\( \zeta_n \)  
\( \omega \)  
\( \lambda \)  
\( \theta \)  
\( \theta \)  
\( \beta \)  
\( \alpha \)  

The coefficient related to the quality of the bond condition and position of the bar during concreting

Curvature when the tension reinforcement first reaches yield strength

Curvature at ultimate when the concrete compression strain reaches a specified limiting value

Damping ratio

Natural circular frequency

Slenderness ratio

Inclination of the compression struts

The compression strut inclination (\( \theta = 45^\circ \) in seismic design)

The lower bound factor for the horizontal design spectrum

Inclination of the stirrups
Preface

This thesis was performed with help and guidance of structural engineering company KE-guppen AB and the Department of Civil and Architectural Engineering, division of Concrete Structures, at Royal Institute of Technology KTH. The thesis was done from January to June 2014 under the supervision of Professor Anders Ansell at KTH and CEO Patrik Gatter at KE-gruppen AB.

I want to give my appreciation to Professor Anders Ansell my supervisor and teacher at KTH for introducing me to this project and awaking my interest in Structural Engineering within Concrete Structures through his courses and tremendous teaching. I also want to show my gratefulness for his priceless guidance through this thesis.

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1. Introduction

Knowledge within designing structures with respect to earthquake induced vibrations is relatively limited in Sweden, compared to internationally. The most recent major earthquake in Sweden happened in 6th of August 2012 close to Halmstad with a magnitude of 4.1 on the Richter-scale. Also, in 1904 Scandinavia experienced an earthquake of magnitude 5.5 on the Richter-scale, in Oslofjorden which was felt in Sweden as well [1]. Sweden’s focus on earthquake resistance for structures such as nuclear power plants has increased significantly since the beginning of the 21st century when 34 countries started to participate in a project led by the European Commission to prepare nuclear power plants for extreme circumstances and events such as earthquakes. In early days Sweden didn’t have any specific requirements regarding earthquake when designing power plants. The new earthquake-specific regulations that were presented in 2005 placed demands on the operators to ensure that their nuclear power plants meet the requirements [2]. Some buildings, such as the Turning Torso in Malmö, are designed to meet the requirements of specific earthquake resistance. Turning Torso is designed to withstand a quake of magnitude 7 on the Richter-scale [3]. For other structures designed to meet the earthquake resistance requirements in Sweden, the Öresund bridge can be mentioned, designed to withstand a quake with a magnitude of 5.7 on the Richter-scale [4]. Researches have been conducted to see if earthquake could be a hazard for Swedish dams. For example by Bodare and Kulhanek [5]. Their conclusion was that there is no hazard against dams in south-western Sweden, but for dams in middle and north of Sweden there is a small risk. In the case were dams are founded on soil, they stated that more detailed investigations is needed. This thesis is about the seismic action analysis of concrete structures with focus on nuclear power plants and the design procedure of seismic design on concrete buildings. To be able to demonstrate such analysis a model of a typical, simplified half part of an electrical nuclear building is studied. A finite element approach was used with the software FEM-Design 3D Structure 12.

1.1 Eurocode 8

Eurocode 8 is presented as “Design of structures for earthquake resistance”. For application on design and construction of buildings and civil structures in seismic regions. Eurocode 8 is suited for common structures and but not applicable on special structures such as nuclear power plants, large dams or offshore structures [6]. The purpose of Eurocode 8 is to ensure the following in an earthquake event:

- Human lives are protected
- Damage is limited
- Structures important for civil protection remain operational

The Eurocodes are made as a harmonization of technical specification but there are alternative procedures, values and recommendations concerning classes with notes indicating where the national codes may be used instead. An example is the seismic zone maps and reference ground acceleration in chapter 3.2.1 of the Eurocode 8.

1.2 DNB Handbook

Eurocode clearly cites that nuclear power plants as well as other special structures are beyond the scope of Eurocode 8. Therefore the Swidish Radiation Safety Authority together with Swedish licensees commissioned Scanscot Technology AB arranged a Safety Guide for Nuclear Structures titled “Dimensionering av Nukleära Byggnadskonstruktioner (DNB)”. The DNB is complement to regulations in “Boverkets föreskrifter och allmänna råd om tillämpning av europeiska konstruktionsstandarder”, the Swedish code based on applications of Eurocodes, for Swedish nuclear
power plants. The DNB handbook can be applied on concrete structures for Swedish nuclear power plants as well as lighter structure such as boiling water reactors (BWR) or pressurized water reactors (PWR). The DNB cites that Eurocode 8 is not applicable for nuclear power plants, therefore DNBs instructions for seismic design is taken from ASCE 4-98 [7]. According to the DNB the seismic design for structure, system and components can be done in these three steps:

- Defining the design earthquake
- Identifying the safety functions that must be maintained during an earthquake
- Verify that these safety functions are maintained during and after the earthquake

The safety principle for seismic influence on a nuclear power plant is that the structure, system and components need to keep their function and maintain the reactor in a safe situation, during the maximum design earthquake so called Safe Shutdown Earthquake (SSE). The term SSE is replaced by a more common term DBE. The international Atomic Energy Agency (IAEA) in IAEA Safety Guide recommends that the structure that is classified to withstand earthquake loading, should be able to withstand the effects of a greater earthquake that it should be designed for DBE, so called Designed Extension Earthquake (DEE). It also mentions that a small change on the initial parameter for designing earthquake gives source to an impoverished situation for the structure. According to the DNB the seismic classification for structure, system and components are in three categories; 1, P and N. In this thesis the class P is in focus, its safety functions in the bearing functionality are:

- Maintaining the integrity of the load bearing structure
- Carrying and protecting the system and components with safety function

There are three main methods to verify that the structural capacity of a building will withstand a seismic load:

- Methods based on experience
- Tests
- Calculations and dynamic structural analysis

Methods based on experience can be used for structures that were not designed to withstand seismic loading or structures that are designed for a certain magnitude of seismic loading that should be verified for a higher magnitude. The most common methods of this type are Seismic Qualification Utility Group (SQUG) and Seismic Margin Assessment (SMA). Tests are used for equipments that are hard to verify with other methods, such as electrical components. Tests are also done on shake boards according to prescribed routines. Calculations and dynamic structural analysis is the most useful and dominating method for safety verification of structures. In this thesis this is the method in focus.

1.3 Aims, goals and contents of the thesis

The aim of this thesis is to present the design procedures for primary seismic beams and columns in concrete structures according to Eurocode 8 and to describe and understand the difference between Eurocode 8 and DNB in seismic analysis of nuclear power plants. Evaluation of the use of DNB instead of Eurocode 8 for nuclear structures is the goal of this thesis. This is done by studying a nuclear building from a test project named SMART2013, and comparing maximum responses from the model structure that corresponds to load combinations from DNB and Eurocode 8.
Chapter 2 contains a description of horizontal and vertical spectrums that are used for nuclear structures in Sweden. Loads and load combinations are presented and explained for use with both DNB and Eurocode 8. These load combinations will here be used in the analysis of a nuclear building.

In chapter 3, a procedure of seismic design of concrete structures is presented, according to Eurocode 8. Important codes which are used for basic design of a concrete building and detailed design procedures for primary seismic beams and columns are also presented in this chapter.

A short description of the SMART2013-project and the specimen tested in this project is given in chapter 4. The tested structure is used and analyzed in this thesis.

In chapter 5 the seismic analysis of the studied structure done by the FEM-Design software, is presented. The procedure of finite element modeling is briefly explained and the results from the dynamic analysis are also presented in this section.

An example given to clarify the design of primary seismic beams and columns is presented with calculations in chapter 6, based purely on Eurocode 8. In this chapter the beam subjected to the highest moment and shear force is designed. From the calculations, minimum moment resistance of the column attached to the beam is presented.

Chapter 7 contains a discussion of the results obtained from DNB and Eurocode 8. In this chapter the difference between primary seismic beam and column response from Eurocode 8 and DNB are compared. An overall structure displacement is also studied. Conclusions drawn from the discussions are finally presented in chapter 8. In this chapter recommendations for further research are also presented.
2. Seismic loads
Seismic loads on the structures are related on its elastic response spectrum that is the translation of ground movement expressed with velocity, displacement, acceleration and frequency. In the seismic design other loads are also included which is defined in the load combinations.

2.1 Seismic action
The seismic action on nuclear structures is defined using an elastic response spectrum. The elastic response spectrum chosen for the studies in this thesis is suitable for nuclear facilities in Sweden and was published by Strålsäkerhetsmyndigheten (SSM), former Swedish Nuclear Power Inspectorate (SKI) and with assistance of Vattenfall and EON, former Sydkraft and Oskarshamn Kraftgrupp (OKG) [8].

![Figure 2.1: Horizontal envelope ground response spectra for a typical Swedish hard rock site with damping ratio of 0.005, 0.02, 0.05, 0.07 and 0.10 according to SKI rapport (1992), from [9].]
The Peak Ground Acceleration (PGA) amounts to $a_h=0.1g$ for horizontal spectra and $a_v=0.09g$ for vertical spectra and it is assumed that the damping of the system is 5%. The displacement $D$ is given by:

$$D = A \sin \omega t$$  \hspace{1cm} (2.1)

where $\omega$ is natural circular frequency and $A$ is a constant. A first derivation of the displacement gives the velocity:

$$V = A\omega \cos \omega t$$  \hspace{1cm} (2.2)

And a second derivation gives the acceleration:

$$a = -A\omega^2 \sin \omega t$$  \hspace{1cm} (2.3)
The natural circular frequency $\omega$ can be written as a function of natural cyclic frequency $f$, as:

$$\omega = 2\pi f$$  \hspace{1cm} (2.4)

It is known that for the maximum value of $a$, the maximum value of $\sin \omega t$ will be:

$$\sin \omega t = 1$$  \hspace{1cm} (2.5)

With Eqs. (2.3)-(2.4) the expression for $a$ can be rewritten as:

$$a = -A(2\pi f)^2$$  \hspace{1cm} (2.6)

Which clearly shows that the ground peak acceleration (PGA) value i.e. the maximum value $a$ of the ground, is valid with a maximum value of $f$, since all other parameters involved are constants. Another way to explain the measurement of the PGA value is to imagine a massive and cubic concrete block that is placed on the ground, subjected to a ground motion. Since the massive block will have a significantly small period $T$, by studying the relationship: $f = 1/T$ it is understandable that:

$$T_{\text{block}} \to 0 \hspace{0.5cm} \text{then} \hspace{0.5cm} f_{\text{block}} = \frac{1}{T_{\text{block}}} \to \infty$$  \hspace{1cm} (2.7)

By measuring the acceleration of the concrete block the PGA value will be observed:

$$a_{\text{block}} = -A(2\pi f_{\text{block}})^2 = \text{PGA}$$  \hspace{1cm} (2.8)

With the use of the both horizontal and vertical spectra presented above, horizontal and vertical design spectra for the pseudo acceleration can be created. Here this was made by reading the frequency for each period and finding the corresponding acceleration according to the damping ratio. Design spectra for the pseudo acceleration are shown in Figures 2.3 and 2.4.

*Figure 2.3; Horizontal design spectrum for pseudo acceleration which corresponds to Figure 2.1 with PGA=0.11g.*
2.2 Eurocode 8

Actions which are included in the seismic design according to Eurocode 8 and corresponding load combination are presented in this section.

2.2.1 Vertical actions

According to Eurocode 8 the loads that should be considered in seismic design are loads that act vertically on the structure, other than the seismic load itself. The reason is that these can be transformed into masses. Loads that are considered on the structure are dead load (self weight of the structure) and live load that applies on each level of the structure. The vertical loads that should be taken in to account in seismic design are \( G \) (dead load) and \( Q \) (variable live loads). Dead load \( G \) is determined by the self weight of the structure and according to Eurocode 1, live load is determined due to the category of the building [11]. Since the building that is to be studied in the following is a nuclear facility, the category of the building will be E2 (industrial use). Live loads are summarized in Table 2.1 below.

### Table 2.1: Categories and imposed loads on floors due to storage and industrial use, from [11].

<table>
<thead>
<tr>
<th>Category</th>
<th>Specific use</th>
<th>Example</th>
<th>( q_k ) [kN/m(^2)]</th>
<th>( Q_k ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1</td>
<td>Areas susceptible to accumulation of goods, including access areas</td>
<td>Areas for storage use including storage of books and other documents</td>
<td>7.5</td>
<td>7.0</td>
</tr>
<tr>
<td>E2</td>
<td>Industrial use</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As Eurocode 1 indicates, there is no specific characteristic value of the imposed load for Industrial use. According to Eurocode 1, “loads in industrial areas should be assessed considering the intended use and the equipment which is to be installed. Where equipment such as cranes, moving machinery etc, are to be installed the effects on the structure should be determined in accordance with EN 1991-3” [11]. Eurocode 8 also mentions that special structures, such as nuclear power plants, offshore structures and large dams, are beyond the scope of Eurocode 8 [6]. Therefore, the live load that has been chosen for analysis of the structure is the same live load that was applied on the model in SMART2013-project, see chapter 4.
2.2.2 Load combination for vertical actions

According to Eurocode 8, the inertial effects of the seismic actions shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of action:

\[ \sum G_{k,j} + \sum \psi_{E,i} \cdot Q_{k,i} \]  \hspace{1cm} (2.9)

where \( G_{k,j} \) is the characteristic value of permanent action, \( Q_{k,i} \) is the characteristic value of the accompanying variable action \( i \) and \( \psi_{E,i} \) is the combination coefficient for variable action \( i \). The combination coefficient \( \psi_{E,i} \) is calculated by the following equation:

\[ \psi_{E,i} = \varphi \cdot \psi_{2,i} \]  \hspace{1cm} (2.10)

where \( \psi_{2,i} \) is the factor for quasi-permanent value of a variable action \( i \). Values for \( \varphi \) and \( \psi_{2,i} \) can be taken from Tables 2.2 and 2.3 where the building types are summarized in categories: A-H.

<table>
<thead>
<tr>
<th>Type of variable</th>
<th>Storey</th>
<th>( \varphi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Categories A-C</td>
<td>Roof</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Storeys with correlated occupancies</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>Independently occupied storeys</td>
<td>0.5</td>
</tr>
<tr>
<td>Categories D-F and Archives</td>
<td></td>
<td>1.0</td>
</tr>
</tbody>
</table>
Table 2.3: Recommended values of $\psi$ factors for buildings, from [12].

<table>
<thead>
<tr>
<th>Action</th>
<th>$\psi_0$</th>
<th>$\psi_1$</th>
<th>$\psi_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Imposed loads on buildings</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Category A: domestic, residential areas</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category B: office areas</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category C: congregation areas</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category D: shopping areas</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category E: storage areas</td>
<td>1.0</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td>Category F: traffic area, vehicle weight ≤ 30 kN</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category G: traffic area, 30kN &lt; vehicle weight ≤ 160kN</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category H: roofs</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Snow loads on buildings</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Finland, Iceland, Norway, Sweden</td>
<td>0.70</td>
<td>0.50</td>
<td>0.20</td>
</tr>
<tr>
<td>Remainder of CEN Member States, for sites located at altitude H &gt; 1000 m a.s.l.</td>
<td>0.70</td>
<td>0.50</td>
<td>0.20</td>
</tr>
<tr>
<td>Remainder of CEN Member States, for sites located at altitude H &lt; 1000 m a.s.l.</td>
<td>0.50</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td><strong>Wind loads on buildings</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>0.2</td>
<td>0</td>
</tr>
<tr>
<td><strong>Temperature (non-fire) in buildings</strong></td>
<td>0.6</td>
<td>0.5</td>
<td>0</td>
</tr>
</tbody>
</table>

As seen in the Tables 2.2 and 2.3, there is no category that specifies a value for nuclear power plants. Because of the importance of these kinds of buildings a value that does not significantly decrease the load acting on the structure is preferred. The proper values are amounted to $\psi_{2,i} = 0.9$ and $\varphi = 1.0$.

2.2.3 Load combination for seismic design situation

Effects of actions for seismic design according to can be written as in the following expression [6]:

$$E_d = E\left\{G_{k,j}\mid P; A_{Ed}; \psi_{2,i}Q_{k,i}\right\} \quad j \geq 1; i \geq 1$$  \hspace{1cm} (2.11)

This combination can be expressed as Eq. (2.12).
where $G_{k,j}$ is the characteristic value of permanent action $j$, $P$ is the relevant representative value of a prestressing action, $A_{Ed}$ is the design value of seismic action, $Q_{k,i}$ is the characteristic value of the variable action $i$ and $\psi_{2,i}$ is the factor for quasi-permanent value of a variable action $i$. Design value of seismic action $A_{Ed}$ can also be written as:

$$A_{Ed} = \gamma_1 \cdot A_{Ek}$$

where $A_{Ek}$ is the characteristic value of seismic action. The value for $\gamma_1$ depends on seismic hazard condition of the seismic region and on public safety consideration which is presented in Table 2.4. Eurocode 8 has defined 4 different importance classes for buildings. These are called importance classes I, II, III and IV which are dependent on three main factors [6]:

- Consequences of collapse on human life
- Importance for public safety and civil protection in the immediate post-earthquake period
- Social and economic consequences of collapse

<table>
<thead>
<tr>
<th>Importance class</th>
<th>Buildings</th>
<th>$\gamma_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.8</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging in the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.</td>
<td>1.2</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>1.4</td>
</tr>
</tbody>
</table>

2.3 Dimensionering av Nukleära Byggnadskonstruktioner, DNB

The included loads and corresponding load combination is different in the DNB compared with Eurocode. DNB includes several components that is included in the load combination but the the seismic action is not increased as Eurocode 8. This will be more clear in this section.

2.3.1 Design ground response spectrum

There are two designing situations according to DNB regarding seismic design:

- SSE-Safe Shutdown Earthquake: Exceptional seismic design
- DEE-Designed Extension Earthquake: Very rare seismic design
The term SSE is now replaced by a more general term DBE for the designing earthquake. The designed ground response spectrum to ensure the reactor safety within DBE defines according to SKI Technical Report 92:3, see Figures 2.1 and 2.2. For designing according to DEE, Strålsäkerhetsmyndigheten (SSM) is responsible to give the needed input data [7].

2.3.2 Seismic loads and load combination for seismic situation
Seismic loads according to DBE and DEE are $E_{DBE}$ and $E_{DEE}$ respectively which is classified as accidental loads. DNB corresponding the two designing situations defines load combinations for each of the cases DBE and DEE. It can be seen in Eqs. (2.14)-(2.15) that DNB considers a larger amount of factors in the seismic design situation than the Eurocode 8.

Load combination, DBE

$$1.0D_k + 1.0H_{gw} + 1.0H_{ge} + 1.0\gamma_{p,unfav} \cdot P_p + 1.0\varepsilon_{cs} + 1.0s + 1.0 \cdot \psi_2L + 1.0 \cdot \psi_2S + 1.0 \cdot \psi_2W_q + 1.0 \cdot \psi_2\Delta T + 1.0H_{qw} + 1.0H_{qe} + 1.0M_d + 1.0E_{DBE}$$

(2.14)

Load combination, DEE

$$1.0D_k + 1.0H_{gw} + 1.0H_{ge} + 1.0P_{p} + 1.0\varepsilon_{cs} + 1.0s + 1.0 \cdot \psi_2L + 1.0 \cdot \psi_2S + 1.0 \cdot \psi_2W_q + 1.0 \cdot \psi_2\Delta T + 1.0H_{qw} + 1.0H_{qe} + 1.0M_n + 1.0M_d + 1.0E_{DEE}$$

(2.15)

where $D_k$ is the dead weight, $H_{gw}$ is the water pressure, $H_{ge}$ is the earth pressure, $P_p$ is the pre-stressed force, $\varepsilon_{cs}$ is the shrinkage, $s$ is the settlement, $L$ is the live load, $S$ is the snow load, $W_q$ is the wind load, $\Delta T$ is the climate related temperature load, $H_{qw}$ is the water pressure difference between normal water level and time variable water level, $H_{qe}$ the soil pressure due to movable surface load, $M_n$ is the process related loads during normal operation and shutdown period, $M_d$ is the process related loads during operation disturbance, $E_{DBE}$ is the load due to designed DBE, $E_{DEE}$ is the load due to designed DEE, $\gamma_{p,unfav}$ is the partial factor for pre-stressing actions and $\psi_2$ is the factor for quasi-permanent value of a variable action, which can be taken from Eurocode and EKS8.
3. Seismic analysis according to Eurocode 8

Some important codes that are needed to be considered when designing concrete buildings are presented and explained in this section. The purpose of this chapter is to provide a background and explanation that can be useful to understand the use of instructed codes on seismic design of concrete buildings according to Eurocode 8.

3.1 Criteria for regularity in plan

According to Eurocode 8, buildings in seismic design are categorized into being regular and non-regular, indicated by a criteria that describe the regularity in plan and elevation of the structure. These regularities will influence the allowed simplifications and behavior factor.

Table 3.1; Consequences of structural regularity on seismic analysis and design, from [6].

<table>
<thead>
<tr>
<th>Regularity</th>
<th>Allowed Simplification</th>
<th>Behavior factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plan</td>
<td>Elevation</td>
<td>Model</td>
</tr>
<tr>
<td>Yes</td>
<td>Yes</td>
<td>Planar</td>
</tr>
<tr>
<td>Yes</td>
<td>No</td>
<td>Planar</td>
</tr>
<tr>
<td>No</td>
<td>Yes</td>
<td>Spatial</td>
</tr>
<tr>
<td>No</td>
<td>No</td>
<td>Spatial</td>
</tr>
</tbody>
</table>

According to Eurocode 8, for a building regular in plan some conditions should be satisfied. These are:

- The slenderness i.e the ratio between larger and smaller length of the building, shall not be higher than 4:

  \[
  \lambda = \frac{L_{\text{max}}}{L_{\text{min}}} \leq 4
  \]  
  \hspace{1cm} (3.1)

- Structural eccentricity \( e_0 \) and the torsional radius \( r_x \), at each level, and for each direction of analysis \( x \) and \( y \), shall be in agreement with:

  \[
  e_{0x} \leq 0.30 \cdot r_x
  \]  
  \hspace{1cm} (3.2)

  \[
  r_x \geq l_g
  \]  
  \hspace{1cm} (3.3)

where \( e_{0x} \) is the distance between the center of stiffness and the center of mass, measured along \( x \)-direction, which is normal to the direction of analysis considered, \( r_x \) is the square root of the ratio of the torsional stiffness to the lateral stiffness in \( y \)-direction (“torsional radius”) and \( l_g \) is the radius of gyration of the floor mass in plan. According to Eurocode 8, in multi storey buildings such as the building that is to be studied here, the center of stiffness and the torsional radius can be determined only approximately. Therefore, for classification of structural regularity, a simplification can be made if the following conditions are satisfied:

- All lateral load resisting systems, such as cores, structural walls, or frames, run without interruption from the foundations to the top of the building.
- The deflected shaped of the individual systems under horizontal loads are not very different. This condition may be considered satisfied in the case of frame systems and wall systems.
3.2 Criteria for regularity in elevation
For a building to be satisfied as regular in elevation, conditions in Eurocode 8 shall be fulfilled. In the case of setbacks, additional conditions are applied. Asymmetric preservation of the studied structure implies that the following condition should be fulfilled: “If the setback do not preserve symmetry, in each face the sum of the setbacks at all storey’s shall not be greater than 30% of the plan dimension at the ground floor above the foundation or above the top of a rigid basement” [6], see Figure 3.1.

\[ \frac{L - L_2}{L} \leq 0.30 \]

Figure 3.1: Criteria for regularity of buildings with setback, from [6].

3.3 Structural type of the building
According to Eurocode 8, the structural system of the buildings is defined as follows:

- Wall system
- Frame system
- Dual system
- Frame-equivalent dual system
- Wall-equivalent dual system
- Torsionally flexible system
- Inverted pendulum system

The condition for each one of the mentioned systems is defined according to Eurocode 8 [6]. For the studied structure here the structural type of the building is investigated to wall system due the system in both vertical and lateral directions, resist the loads mainly by structural walls, whose shear resistance at the base exceeds 65% of the total shear resistance of the whole structural system.
3.4 Ductility
When buildings are subjected to strong ground shaking, they are expected to have the ability to deform beyond the limit of linearly elastic behavior. This deformation into the structures inelastic (plastic) range is of central importance in earthquake engineering. The ability to deform and to dissipate energy, without a substantial reduction in strength is called “Ductility”. Figure 3.2 shows the difference between linearly elastic and elastoplastic systems regarding their peak deformation, due to e.g. earthquake ground motion. Both systems have the same stiffness, mass and damping [13]. Figure 3.2 clearly show that an elastoplastic system can undergo much larger deformations than its corresponding linear elastic system after reaching its yielding point. According to Eurocode 8, in seismic design of concrete buildings, structures are classified in three ductility classes DCL (low ductility), DCM (medium ductility) and DCH (high ductility). Design with DCL is recommended only in low seismic cases. Otherwise concrete buildings that are designed to resist earthquake, shall provide energy dissipation capacity and an overall ductile behavior. To be able to achieve this behavior, Eurocode classifies ductility classes into two categories of DCM and DCH.

![Figure 3.2: Elastoplastic system and its corresponding linear system, from [13].](image)

3.5 Behavior factors for horizontal seismic action
In accordance with Eurocode 8, the behavior factor $q$ is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping. The seismic forces used in the design, is the input for a conventional elastic analysis model, that ensures a satisfactory response of the structure. The upper limit of the factor $q$, to account for the energy dissipation capacity, is derived as it is shown in Eq. (3.4).

$$q = q_0 \cdot k_w \geq 1.5$$  \hspace{1cm} (3.4)

Yielding stress.
Yield deformation.
Maximum deformation.
Peak value of the earthquake-induced resisting deformation.
Peak value of the earthquake-induced resisting force.
where $q_0$ is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation and $k_w$ is the factor reflecting the prevailing failure mode in structural systems with wall. The basic value of the behavior factor $q_0$ is given in Table 3.2, depending on the systems ductility class:

<table>
<thead>
<tr>
<th>Structural type</th>
<th>DCM</th>
<th>DCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame system, dual system, coupled wall system</td>
<td>$3.0\alpha_u/\alpha_l$</td>
<td>$4.5\alpha_u/\alpha_l$</td>
</tr>
<tr>
<td>Uncoupled wall system</td>
<td>$3.0$</td>
<td>$4.0 \alpha_u/\alpha_l$</td>
</tr>
<tr>
<td>Torsionally flexible system</td>
<td>$2.0$</td>
<td>$3.0$</td>
</tr>
<tr>
<td>Inverted pendulum system</td>
<td>$1.5$</td>
<td>$2.0$</td>
</tr>
</tbody>
</table>

Table 3.2: Basic value of the behavior factor, $q_0$, for systems regular in elevation. For systems which are not regular in elevation, the value $q_0$ should be reduced by 20%, from [6].

where $\alpha_l$ is the value by which the horizontal seismic design action is multiplied, in order to first reach the flexural resistance in any member of the structure, while all other design actions remain constant and $\alpha_u$ is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. Eurocode 8 indicates an approximation value of $\alpha_u/\alpha_l$ for buildings which are regular in plan. The factor reflecting the prevailing failure mode in structural system is calculated depending on its structural system. Eq. (3.5) shows the relation for frame and frame-equivalent dual systems while Eq. (3.6) shows the relation for wall, wall-equivalent and torsionally flexible systems.

\[
k_w = 1.00 \quad (3.5)
\]

\[
k_w = 0.5 \leq \frac{1 + \alpha_0}{3} \leq 1 \quad (3.6)
\]

The factor $\alpha_0$ is defined as the prevailing aspect ratio of the walls of the structural system. According to Eurocode 8, if the aspect ratios $h_w/l_{wi}$ of all walls $i$ of the structure does not differ significantly, $\alpha_0$ can be calculated by Eq. (3.7).

\[
\alpha_0 = \frac{\sum h_{wi}}{\sum l_{wi}} \quad (3.7)
\]

where $h_{wi}$ is the height of the wall $i$ and $l_{wi}$ is the length of the section of wall $i$. By applying the behavior factor to the horizontal and vertical spectra from SKI: report 1992 which are shown in Figures 2.1 and 2.2, a relatively smaller design spectrum is obtained. First the equations for design response spectra both for vertical and horizontal envelopes which are presented in Eurocode 8 is studied. These equations are influenced by the behavior factor as it is expressed in Eqs. (3.7)-(3.10). Figure 3.3 illustrate the different periods for a typical shaped elastic response spectrum which are used in Eqs. (3.8)-(3.11).
Figure 3.3: A typical shaped elastic response spectrum showing different periods, from [6].

\[
0 \leq T \leq T_B: S_d(T) = a_g \cdot S \cdot \left[ \frac{2}{3} + \frac{T}{T_B} \cdot \left( \frac{2.5}{q} - \frac{2}{3} \right) \right] \\
\quad (3.8)
\]

\[
T_B \leq T \leq T_C: S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \\
\quad (3.9)
\]

\[
T_C \leq T \leq T_D: S_d(T) = \begin{cases} 
  a_g \cdot S \cdot \frac{2.5}{q} \left[ \frac{T_C}{T} \right] \\
  \geq \beta \cdot a_g 
\end{cases} \\
\quad (3.10)
\]

\[
T_D \leq T: S_d(T) = \begin{cases} 
  a_g \cdot S \cdot \left[ \frac{T_C \cdot T_D}{T^2} \right] \\
  \geq \beta \cdot a_g 
\end{cases} \\
\quad (3.11)
\]

where \(a_g\) is the design ground acceleration, \(S\) is the soil factor; see Eurocode 8 Table 3.1, \(T_B\) is the lower limit of the period of the constant spectral acceleration branch, \(T_C\) is the upper limit of the period of the constant spectral acceleration branch, \(T_D\) is the value defining the beginning of the constant displacement response range of the spectrum, \(S_d(T)\) is the design spectrum, \(q\) is the behavior factor and \(\beta\) is the lower bound factor for the horizontal design spectrum which is recommended to 0.2 according to Eurocode 8 [6].
3.6 Methods of analysis

There are four methods of analysis possible for determination of the seismic effects on a structure according to [6]:

- Lateral force method of analysis.
- Modal response spectrum analysis.
- Non-linear static (pushover) analysis.
- Non-linear time history (dynamic) analysis.

Method of analysis is chosen depending on the structures characteristics, see Table 3.1. The characteristics of the studied structure, according to Table 3.1, indicate that the proper method of analysis to determine the seismic effects is “Modal response spectrum analysis”.

3.6.1 Modal response spectrum analysis

According to Eurocode 8 all modes of vibration that considerably contribute to the global response shall be taken into account, which may be deemed to be fulfilled if the two conditions below can be demonstrated:

- The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure.
- All modes with effective modal masses greater than 5% of the total mass are taken into account.

Eurocode 8 indicates that if these two conditions cannot be satisfied for each relevant direction, a minimum number of modes $k$ shall be taken into account, fulfilling Eqs. (3.11)-(3.12):

$$k \geq 3 \cdot \sqrt{n}$$

$$T_k \leq 0.20 \text{ sec}$$

where $k$ is the number of modes taken into account, $n$ is the number of storeys above the foundation/the top of a rigid basement and $T_k$ is the period of vibration of mode $k$. The maximum value of a seismic action effect can be calculated with two different methods, SRSS (Square Root of Sum of Squares) and CQC (Complete Quadric Combination), depending on the following condition according to FEM Design theory book:

$$\begin{cases} T_j \leq 0.9 \cdot T_i \\
\text{Otherwise} \end{cases} \quad \text{SRSS: } E_E = \sqrt{\sum E_{Ei}^2}$$

$$\quad \text{CQC: } E_E = \sqrt{\sum \sum E_{Ei} \cdot r_{ij} \cdot E_{Ej}}$$

With $T_j \leq T_i$
where $i$ and $j$ are the vibration modes including both translational and torsional modes, $T_j$ and $T_i$ are the vibration periods of mode $i$ and $j$, $E_E$ is the seismic action effect under consideration, $E_{EI}$ and $E_{EJ}$ are the value of the considered seismic action effect on the vibration mode $i$ and $j$ and $r_{ij}$ is the interaction between two vibration periods taking into account the declining ratio. The interaction $r_{ij}$ can be determined by Eq. (3.15) [14]:

$$
 r_{ij} = \frac{\xi^2(1+r)^{3/2}}{(1-r)^2+4\xi^2r(1-r)^2}
$$

(3.15)

where $r = T_j/T_i$. Torsional effects are, according to Eurocode 8, calculated by the following expression. The floor dimension on the storey $i$ will be obtained for each direction as shown in Figure 3.4.

$$
 M_{ai} = e_{ai} \cdot F_i
$$

(3.16)

$$
 e_{ai} = \pm 0.05 \cdot L_i
$$

(3.17)

where $M_{ai}$ is the torsional moment applied at storey $i$ about vertical axis, $F_i$ is the horizontal force acting on storey $i$, $e_{ai}$ is the accidental eccentricity of storey mass $i$ and $L_i$ is the floor dimension perpendicular to the direction of the seismic action.

*Figure 3.4; Explanation of the floor dimension on the $i^{th}$ storey, from [15].*
3.7 Design for DCH
In the studied case, the structure is for a nuclear facility and Eurocode does not give any recommendation regarding what type of ductility class has to be chosen. The engineer should choose the best possible solution for the structure that is to be designed. Depending on the type of building and its importance to deform beyond its elastic range during an earthquake, a high ductility class is chosen, DCH.

3.7.1 Material requirements
Primary seismic members of the structure shall not have a concrete strength class lower than C20/25. Eurocode 8 also recommends that in critical regions of primary seismic elements of the structure, strength class of the reinforcement that should be used is Class C. Properties of reinforcement steel classes are shown in Table 3.3 below. These properties are valid within temperatures between -40 °C and +100°C in the finished structure.

Table 3.3: Properties of reinforcement, from [16].

<table>
<thead>
<tr>
<th>Product form</th>
<th>Bars and de-coiled rods</th>
<th>Wire Fabrics</th>
<th>Requirement or quantile value [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class</td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Characteristic yield strength $f_{yk}$ or $f_{0.2k}$ [Mpa]</td>
<td>400 to 600</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>Minimum value $k=(f/t)_{yk}$</td>
<td>≥0.5</td>
<td>≥1.08</td>
<td>≥1.15</td>
</tr>
<tr>
<td>Characteristic strain at maximum force, $\varepsilon_{uk}$ [%]</td>
<td>≥2.5</td>
<td>≥5.0</td>
<td>≥7.5</td>
</tr>
<tr>
<td>Bendability</td>
<td>Bend/Rebend test</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Shear strength</td>
<td>0.25 $Af_{yk}$</td>
<td>Minimum</td>
<td></td>
</tr>
<tr>
<td>(A is area of wire)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum deviation from nominal mass(individual bar or wire) [%]</td>
<td>≤8</td>
<td>±6.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt;8</td>
<td>±4.5</td>
<td></td>
</tr>
</tbody>
</table>

3.7.2 Geometrical constraints
According to Eurocode 8, the primary seismic beams should have at least a width of 200 mm. Also, it is mentioned that the width to height ratio of these beams should satisfy the Eq. (3.18).
\[
\frac{l_{ot}}{b} \leq \frac{70}{\left(\frac{h}{b}\right)^{1/3}}; \quad \frac{h}{b} \leq 3.5
\]

(3.18)

where \(l_{ot}\) is the distance between torsional restraints, \(h\) is the total depth of beam in central part of \(l_{ot}\) and \(b\) is the width of compression flange. Eurocode 8 indicates that the distance between the centroidal axis of two members should not exceed \(b_c/4\). Where \(b_c\) is the largest cross-sectional dimension of the column normal to the beam axis. Another requirement for the beam is that the width of the primary seismic beam \(b_w\), shall satisfy the expression below, account for the effect of the bars that are passing through the joint:

\[
b_w \leq \min\{b_c + h_w; 2b_c\}
\]

(3.19)

According to Eurocode 8, the cross-section of the columns should not have a dimension that is less than 250 mm. Eurocode 8 point out that the thickness of the web \(b_{wo}\) should satisfy the following expression:

\[
b_{wo} \geq \max\{0.15, h_s/20\}
\]

(3.20)

where \(h_s\) is the storey height in meters. According to Eurocode 8, irregularities on forming the coupled walls and random openings should be avoided. These can be done if the influence is either insignificant or included in the analysis, dimensioning and detailing.

3.7.3 ULS verifications and detailing of beams

According to Eurocode 8 calculation for bending and shear resistance of a beam subjected to an earthquake loading, shall be performed in accordance with EN 1992-1-1:2004, unless specified in Eurocode 8 [6]. The effective flange width is the area that a part of the top reinforcement of a primary seismic beam can be placed outside the width of the web. The effective flange width for primary seismic beams both framing into exterior and interior column, with and without transverse beams, is shown in Figures 3.5 and 3.6. The interior and exterior beam-column joint is presented in Figure 3.7.
Shear resistance

Shear resistance is calculated in accordance with and shall be performed as classified in EN 1992-1-1:2004 [6]. Additional conditions that has to be considered for designing the primary seismic beam, regarding shear resistance, is explained in Eurocode 8. Critical region of a primary seismic beam is a...
region of the beam that most likely will yield during a seismic loading of the structure. These regions are located closer to an end cross-section where a beam is framed into a beam-column joint or any other cross section. The critical region \( l_{cr} \), is calculated as in Eurocode 8, by the following expression:

\[
l_{cr} = 1.5h_w
\]  

(3.21)

where \( l_{cr} \) is the length of the critical region from the connecting joint and \( h_w \) is the height of the beam. As can be seen in Figure 3.8, the maximum distance between the column-beam joint and the last shear reinforcement should not be more than or equal to 50 mm.

![Figure 3.8: Transverse reinforcement in critical regions of beams according to Eurocode 8 [6].](image)

**Local ductility satisfaction for shear resistance**

Ductility of a structure, as mentioned earlier, can be defined as the ability of the structure to deform beyond the limit of linearly elastic behavior and to dissipate energy, without a substantial reduction in strength. Some factors that can influence this deformability are the tensile reinforcement ratio, the amount of longitudinal compressive reinforcement, the amount of lateral tie and the strength of the concrete. The reinforced concrete sections ductility may be chosen as the curvature ductility \( \mu_\theta \) [18]:

\[
\mu_\theta = \frac{\phi_u}{\phi_y}
\]  

(3.22)

where \( \phi_u \) is the curvature at ultimate when the concrete compression strain reaches a specified limiting value and \( \phi_y \) is the curvature when the tension reinforcement first reaches yield strength. However Eurocode 8 introduce another form of calculation for curvature ductility factor which is based on the relationship between \( \mu_\phi \) and the displacement ductility factor \( \mu_\delta \), which is a conservative approximation for concrete members according to Eurocode 8, see Eq. (3.23).

\[
\mu_\phi = 2\mu_\delta - 1
\]  

(3.23)
The relationship between $\mu_\delta$ and $q$, $\mu_\delta = q$, is considered. $q_0$ has a higher value than $q$ in irregular buildings and therefore $q_0$ is used instead of $q$ in determination of the curvature ductility factor. To satisfy the local ductility requirements in Eurocode 8, value of the curvature ductility factor $\mu_\phi$ should satisfy one of the expressions showed in Eq. (3.24) dependent on the relation between $T_1$ and $T_C$.

$$
\begin{cases}
\mu_\phi = 2q_0 - 1 & T_1 \geq T_C \\
\mu_\phi = 1 + \frac{2(q_0 - 1)T_C}{T_1} & T_1 < T_C
\end{cases}
$$

(3.24)

where $q_0$ is the basic value of the behavior factor, $T_1$ is the fundamental period of the building within vertical plane where bending takes place, $T_C$ is the period at the upper limit of the constant acceleration. When designing the reinforcement an important factor is that in critical regions the strut inclination $\theta$ in the truss model shall be 45°. This will give the lower limit of $\cot \theta = 1$. This inclination will influence the shear load capacity $V_{rd,s}$ and the so called web compression failure $V_{rd,max}$. The following expressions show that decreasing value of $\cot \theta$, will result in a decreasing value for $V_{rd,s}$ and an increasing value for $V_{rd,max}$:

$$
V_{rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot (\cot \theta + \cot \alpha) \sin \alpha
$$

(3.25)

$$
V_{rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v \cdot f_{cd} \cdot \frac{\cot \theta}{1 + \cot^2 \theta}
$$

(3.26)

where $\theta$ is the inclination of the compression struts, $\alpha$ is the inclination of the stirrups, $s$ is the distance between stirrups, $A_{sw}$ is the cross-sectional area of one stirrup, $f_{ywd}$ is the yield strength of the shear reinforcement and $v$ is the reduction factor; $v = 0.6 \cdot (1 - f_{ck}/250)$. From Eqs. (3.25)-(3.26) it is clear that Eurocode 8, put more weight on $V_{rd,s}$ i.e. shear strength regarding the reinforcement. The ratio $\xi$ which is introduced in Eurocode 8 is a relationship between the maximum $V_{Ed,max}$ and minimum $V_{Ed,min}$ shear force in section $i = 1, 2$ denoting the end sections of the beam. Dependent on this ratio the shear resistance computation type will be determined. If $\xi \geq -0.5$ The computation should be followed by the procedures introduced in Eurocode 2 [14]. For local ductility to be satisfied, some special rules for shear reinforcements, i.e. hoops, in the critical regions of the primary seismic beams, should be taken into account according to Eurocode 8 [6]:

- Minimum diameter $d_{bw}$ of the bars used shall be 6 mm
- Spacing of the hoops $s$ should satisfy Eq. (3.27).

$$
s = \min \left\{ \frac{h_w}{4} ; 24d_{bw} ; 175 ; 6d_{bl} \right\}
$$

(3.27)

where $h_w$ is the depth of the beam, $d_{bw}$ is the diameter of the hoops ($d_{bw} \geq 6mm$), $d_{bl}$ is the minimum longitudinal bar diameter.
Shear force carrying capacity without shear reinforcement

A concrete beam is able to take shear forces without having any shear reinforcement. Instead shear forces will be carried partly by the concrete itself and partly by the flexural reinforcement in the beam. To clarify how the load can be carried by the beam without shear reinforcement a cracked section of a concrete beam with flexural reinforcement can be studied in Figure 3.9.

![Figure 3.9: Forces which acts on a beam lamella and development of a shear crack, from [19].](image)

As seen in Figure 3.9, $V_1$ is the shear force carried by the concrete itself in the compression zone, where concrete is able to take up the shear stresses. $V_3$ is from friction and interlocking action between concrete surfaces. When the developed cracks reach a certain width, the interlocking and friction action between the concrete surfaces will decrease, thus the shear force $V_3$ will instead be transmitted to the bending reinforcement as a concentrated shear force, a so called dowel force. It is also important to mention that the increased longitudinal reinforcement content influence the shear force carrying capacity of a beam without shear reinforcement [20], so that:

- If the height of the compression zone increases, the ability to carry shear force $V_1$ is increased.
- Increasing reinforcement makes it more difficult for the cracks to open, thus more friction action is obtained between the concrete surfaces and the ability to carry $V_2$ increases.
- If the transversal displacement of the beam decreases, the dowel forces $V_3$ increases.

The expression for the shear force carrying capacity $V_{rd,c}$ of a beam without shear reinforcement in Eurocode 2 is purely empirical. Thus a mathematical model with the parameters that influence the load carrying capacity was established in a way that it corresponds with a large number of test results in a best possible accordance. See Eq. (3.28) for the mechanical model.

$$V_{rd,c} = \left(0.18 \cdot \frac{k \cdot 2\sqrt{100} \cdot \rho_1 \cdot f_{ck} + 0.15\sigma_{cp}}{\gamma_c} \right) \cdot b_w d$$  \hspace{1cm} (3.28)

where $\gamma_c = 1.5$ is the partial coefficient for concrete strength, $k = 1 + \sqrt{200/d} \leq 2.0$ is the factor which considers the effective height, $\rho_1 = A_{st}/b_w d \leq 0.02$ is the reinforcement content with area $A_{st}$, $f_{ck}$ is the characteristic compressive stress of concrete, $\sigma_{cp} = N_{Ed}/A_c$ is the average compressive stress, $N_{Ed}$ is the normal force from tensioning or external pressure, $A_c$ is the concrete area of the cross-section, $b_w$ is the smallest width of the cross-section with the tensile zone and $d$ is the effective height. It is seen in the expression for $V_{rd,c}$ that if $\rho_1 \to 0$ then $V_{rd,c} \to 0$. But the
concrete itself reinforcement is also able to carry shear force which is considered the minimum shear force capacity of the concrete $v_{\text{min}}$. It can be calculated by the following expression:

$$v_{\text{min}} = 0.035 \sqrt{k^3 f_{ck}}$$

(3.29)

Amount of shear reinforcement

If the shear force acting on a beam without shear reinforcement exceeds the shear resistance, the beam should be reinforced for the remaining shear forces that cannot be carried. The shear reinforcement needed can be calculated as [21]:

$$\frac{A_{sw}}{s} = \frac{V_1}{f_{ywd} \cdot z \cdot (\cot \theta + \alpha) \sin \alpha}$$

(3.30)

where $V_1$ is the force that is needed to be taken by shear reinforcement, $\alpha$ is the stirrup inclination, $\theta$ is the compression strut inclination ($\theta = 45^\circ$ in seismic design), $A_{sw}$ is the cross sectional area of one stirrup, $s$ is the distance between stirrups and $f_{ywd}$ is the yield strength of the shear reinforcement.

According to Eurocode 2, in design of shear reinforcement, in a region where shear force $V$ does not have discontinuity, the designing shear force within the acceptable region with length $z \cdot \cot \theta$ is the least value of $V$ in that region. This can be seen more clearly in Figure 3.10. In seismic design situation for nuclear power plants, there is no recommendation in Eurocode about how to design the shear reinforcements. Therefore, it is highly recommended to use the maximum value of the shear force on the regions needed to be reinforced.

![Figure 3.10; Design of shear reinforcement, from [22].](image)
Bending resistance

According to Eurocode 8, the bending resistance shall be designed as described in Eurocode 2 [14]. Eurocode 8 additionally mentions that the effective flange width \( b_{\text{eff}} \) shall be calculated as was mentioned here in the beginning of section 3.7.3. The maximum bending moment was calculated with maximum value of the load combinations by FEM-Design software. In accordance to this calculation, the computation is performed for designing the bending resistance. When designing a beam for bending with known geometry, concrete class, steel class, \( M \) and \( N \) values, first the compression zone height \( x \) is determined and then the calculation for reinforcement area is performed. The compression zone is calculated as:

\[
\frac{x}{d} = \frac{1}{2\beta} - \frac{1}{\sqrt{4\beta^2 - \frac{M}{\alpha f_{\text{cd}} b d^2}}} \tag{3.31}
\]

When the compression zone height \( x \) is determined, the reinforcement area can be calculated using the following expression from cross-sectional force equilibrium [23]:

\[
f_{yd} A_s - \alpha f_{\text{cd}} b x = 0 \tag{3.32}
\]

\[
A_s = \frac{\alpha f_{\text{cd}} b x}{f_{yd}} \tag{3.33}
\]

Anchorage length of the longitudinal bars is calculated in accordance with Eurocode 2. The anchorage shall be designed in a way that the bond forces transmit to the concrete so that longitudinal cracking or spalling is prevented. To prevent bond failure the value of the ultimate bond stress must be adequate. Eurocode 2 has an expression for the design value for the ultimate bond stress \( f_{\text{bd}} \) with ribbed bars [14]:

\[
f_{\text{bd}} = 2.25 \eta_1 \eta_2 f_{\text{ctd}} \tag{3.34}
\]

where \( f_{\text{ctd}} \) is the design value of concrete tensile strength, \( \eta_1 \) is the coefficient related to the quality of the bond condition and position of the bar during concreting and \( \eta_2 \) is the coefficient related to the bar diameter. As mentioned, the coefficient \( \eta_1 \) depends on the position of the bars during concreting and can be taken from Eurocode 2. For the coefficient \( \eta_2 \) the value depend on the bar diameter, according to Eurocode 2 [14]:

\[
\eta_2 = \begin{cases} 
1.0 & \text{if } \varnothing \leq 32\text{mm} \\
\frac{132 - \varnothing}{100} & \text{if } \varnothing > 32\text{mm} 
\end{cases} \tag{3.35}
\]

The required anchorage length depends on the type of steel in use and the ultimate bond stress. Eq. (3.36) can be used for determining the required anchorage length.
where \( \sigma_{sd} \) is the stress corresponding to the design value. The design anchorage length can, according to Eurocode 2, be calculated by the Eq. (3.36) where \( \alpha_1 \) is the coefficient for Effect of bar form assuming adequate cover, \( \alpha_2 \) is the coefficient for Effect of concrete minimum cover, \( \alpha_3 \) is the coefficient for Effect of confinement by transverse reinforcement, \( \alpha_4 \) is the coefficient for Effect of welded transverse bars, \( \alpha_5 \) is the coefficient for Effect of the pressure transverse to the plane of splitting along design anchorage length and \( l_{b,min} \) is the minimum anchorage length [14].

\[
l_{b,rqd} = \frac{\sigma_{sd}}{4f_{bd}} \quad (3.36)
\]

\[
l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,min} \quad (3.37)
\]

Coefficients \( \alpha_2, \alpha_3, \alpha_4 \) and \( \alpha_5 \) can be taken from Eurocode 2. The minimum length that should be provided to anchorage the reinforcements is called minimum anchorage length \( l_{b,min} \) and is calculated as [14]:

\[
l_{b,min} = \begin{cases} \max(0.3l_{b,rqd}, 100 \text{ mm}) & \text{for tensioned bar} \\ \max(0.6l_{b,rqd}, 100 \text{ mm}) & \text{for compressed bar} \end{cases} \quad (3.38)
\]

According to Eurocode 8, to satisfy the local ductility in seismic design, at least two bars with 14 mm in diameter has to be provided at top and bottom of the beam that goes along the length of the beam. Another rule that needs to be considered is that \( \frac{1}{4} \) of the maximum top reinforcement placed at the supports must be provided along the length of the beam.

### 3.7.4 ULS verifications and detailing of columns

According to Eurocode 8, the design and dimensioning of columns in seismic design is done by the procedure explained in Eurocode 2 [14]. Similar to what was mentioned for primary seismic beams, primary seismic columns critical region is where most likely to yield during seismic loading. The critical region is computed by the following expression (in meters) [6]:

\[
l_{cr} = \max\{1.5h_c; \frac{l_{cl}}{6}; 0.6\} \quad (3.39)
\]

where \( h_c \) is the largest cross-section of the column and \( l_{cl} \) is the clear length of the column. In accordance with Eurocode 8 in a situation where \( l_{cl}/6 < 3 \), the entire length of the primary seismic beam is considered critical. In the first two stories of the buildings the critical region \( l_{cr}^* \) is computed as follows in Eq. (3.40).

\[
l_{cr}^* = l_{cr} + 0.5l_{cr} = 1.5l_{cr} \quad (3.40)
\]

where \( l_{cr}^* \) is the critical region of the first two storeys.
**Longitudinal reinforcement**

Eurocode 2 indicates that the recommended minimum diameter of bars used as longitudinal reinforcement is 8 mm. This is logical if an uncracked concrete beam subjected to loading is considered. By the time the beam starts to crack, if the bars are not strong enough, the risk for local brittle failure of the reinforcement is large. The idea is to transfer the load directly to the reinforcement immediately after cracking, and bars should be strong enough to take up the transferred forces. The same can be applied for columns. Bars with a diameter less than 8 mm can be too weak for such a transfer. The column corners (rectangular or polygon) shall be at least provided with one intermediate bar that goes along the columns height, to make certain the integrity of the joint between column and the beam. In the case of circular cross sections there is no recommendation in Eurocode 8, but it is mentioned in Eurocode 2 that the column with circular cross section shall have minimum four longitudinal bars with a minimum and maximum reinforcement ratio that is 0.01 and 0.04, respectively. Eurocode 8 indicates that the distance between consecutive longitudinal bars that are restrained by transverse reinforcement (hoops), shall not exceed 150 mm. It is also mentioned that the quantity of the longitudinal bars used in the bottom of the column, at the base level where column is connected to foundation, shall not vary from the top of the column [14].

**Transverse reinforcement**

To ensure the minimum ductility and to prevent the local buckling of the longitudinal reinforcement, a transverse reinforcement of minimum 6 mm shall be provided. This can be deemed satisfied if Eqs. (3.40)-(3.41) are considered:

\[
d_{bw} \geq 0.4d_{bl,max} \sqrt{\frac{f_{ydl}}{f_{ydw}}} \quad (3.40)
\]

\[
s = \min\{b_o/3; 125; 6d_{bl}\} \quad (3.41)
\]

where \(d_{bw}\) is the minimum dimension of the longitudinal bars, \(d_{bl}\) is the minimum dimension of the transverse bars and \(b_o\) is the minimum dimension of the concrete core (to the inside of the hoops). Normalized axial force \(v_d\) according to Eurocode 8 for DCH design shall not exceed 0.55, which can be calculated by the following expression:

\[
v_d = \frac{N_{Ed}}{A_c f_{cd}} \quad (3.42)
\]

In multi storey buildings it is important to prevent soft story plastic mechanisms by increasing the columns resistance with respect to the beam resistance. According to Eurocode 8 the mechanism can be prevented by the satisfaction of the Eq. (3.43).

\[
\sum M_{Rc} \geq 1.3 \sum M_{Rb} \quad (3.43)
\]
where $\sum M_{Rc}$ is the sum of design values of the moments of resistance of the columns framing the joint and $\sum M_{Rb}$ is the sum of design values of the moments of resistance of the beams framing the joint. According to Eurocode 8, wherever the column is protected against plastic hinging that results from the soft story plastic mechanism, the value $q_0$ may be replaced by $2/3 q_0$. Soft storey plastic mechanisms has led to many building failures during earthquake events. This mechanism can be expected in buildings that needs more space on the ground floor, such as buildings with parking garages on the base floor or buildings with shops on the ground floor. These kinds of buildings usually are configured as shown in Figure 3.11:

![Figure 3.11: Examples on buildings that can lead to soft storey plastic mechanism; Upper stories with infill masonry, longer columns at the ground level and discontinuous columns, from [24].](image)

Therefore, the ground storey will have a lower stiffness relative to the upper stories. This makes most commercial buildings vulnerable against earthquakes. By studying a simple 2D structure and considering the columns to have a fixed-fixed boundary condition, the stiffness of the storey is calculated with the model that is presented in Figure 3.12, subjected to a unit displacement $u=1$:

![Figure 3.12: Storey stiffness of a structure due to dynamic loading with a unit displacement of $u=1$ according to Karoumi [25].](image)
The stiffness of the frame in Figure 3.12 can be expressed as:

\[
k = \frac{2AEI}{L^3}
\]  

(3.44)

Eq. (3.44) clearly shows that with increasing column length the stiffness will decrease significantly. This is one of the reasons that can lead to a soft storey mechanism failure, another is the variation between the stiffness of the floors that may form plastic hinges. This can be more clearly illustrated in Figure 3.13 where the upper stories act as a large stiff mass.

![Model representation of a building with Soft Storey Mechanism on the ground floor due to stiffness variation, from [24].](image)

As an example, the failure of a six storey building during the year 2008 China earthquake, with the magnitude of 7.9 on the Richter scale, can be mentioned [26]. The soft storeyed ground floor formed plastic hinges at the column ends of all the columns placed on this floor. This is shown in Figures 3.14 and 3.16.

![Formation of plastic hinges due to “soft storey mechanism” failure, from [24].](image)
To satisfy the local ductility Eq. (3.45) shall be satisfied according to Eurocode 8 where $\omega_{wd}$ is the mechanical volumetric ratio of confining hoops within the critical region, $\mu_\psi$ is the required value of the curvature ductility factor, $v_d$ is the normalized design axial force, $\varepsilon_{sy,d}$ is the design value of tension steel strain at yield, $b_c$ is the gross cross-sectional width, $b_o$ is the width of confined core (to the centerline of the hoops) and $\alpha$ is the confinement effectiveness factor.

$$\alpha \omega_{wd} \geq \mu_\psi v_d \varepsilon_{sy,d} \frac{b_c}{b_o} - 0.035$$ (3.45)

The mechanical volumetric ratio of confining hoops within the critical region $\omega_{wd}$ is calculated according to:

$$\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \frac{f_{yd}}{f_{cd}}$$ (3.46)
The minimal value for $\omega_{wd}$ that should be provided within the critical region at the base of the column and the critical regions above the base of the column are 0.12 and 0.08, respectively. The confinement effectiveness factor $\alpha$ is calculated as:

$$\alpha = \alpha_n \alpha_s$$  \hspace{1cm} (3.47)

<table>
<thead>
<tr>
<th>$\alpha_i$/Cross section</th>
<th>Rectangular</th>
<th>Circular with circular hoops</th>
<th>Circular with spiral hoops</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_n$</td>
<td>$1 - \sum_{i} \frac{b_i^2}{6b_o h_o}$</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>$\alpha_s$</td>
<td>$(1 - \frac{s}{2b_o})(1 - \frac{s}{2h_o})$</td>
<td>$(1 - \frac{s}{2D_o})^2$</td>
<td>$(1 - \frac{s}{2D_o})$</td>
</tr>
</tbody>
</table>

where $n$ is the total number of longitudinal bars laterally engaged by hoops or cross ties, $b_i$ is the distance between consecutive engaged bars $\leq 150$ mm, $D_o$ is the diameter of confined core (to the centerline of hoops) and $h_o$ is the depth of confined core (to the centerline of hoops).

![Figure 3.17](image-url)
4. Studied three storey structure

The model that is to be studied is taken from the SMART2013-project [26] which was supported by Commissariat à l’Energie Atomique et aux Energies Alternatives (CEA) and Electricité de France (EDF). The first part of the project was completed in year 2007, studying conventional design methods for structural dynamic response and floor response spectra calculations. Also best-estimate methods for structural dynamic response and floor response spectra evaluation were compared. To improve the knowledge another research program started in 2011 named, SMART2013-project. Three new approaches are that the input signals are real and not synthetic, high intensity seismic loadings are applied at the beginning of the testing sequence and the effect of an aftershock is considered.

4.1 The SMART2013 model

The original structure was designed by IOSIS Industries. The studied model is a trapezoidal, three story reinforced concrete structure that represents a typical, simplified half part of an electrical nuclear building, as shown in Figure 4.1. The motivation for its trapezoidal shape is that this makes it possible to study the torsional behavior of the structure [26].

Figure 4.1; Trapezoidal; three story reinforced concrete structure that represents a typical, simplified half part of an electrical nuclear building, from [27].
This model will be modeled using FEM-Design software from which the natural frequencies of the tested model will be compared to the numerical results for verification. Thereafter, the seismic analysis will be done for the structure, with the Swedish response spectra.

4.2 Structural drawings
The plan and elevation drawings of the 1/4\textsuperscript{th} scaled structure from the SMART2013-project are presented in Figures 4.2-4.7. Note that the structure that is to be modeled will have the full scaled dimensions, i.e. the dimension shown in the Figures multiplied by 4.

![Plan drawing of the 1/4\textsuperscript{th} scaled SMART2013-project structure at level ±0.000, from [28].](image-url)

Figure 4.2; Plan drawing of the 1/4\textsuperscript{th} scaled SMART2013-project structure at level ±0.000, from [28].
Figure 4.3: Plan drawing of the 1/4th scaled SMART2013-project structure at level +1.250 - +2.450, from [28].
Figure 4.4: Plan drawing of the 1/4th scaled SMART2013-project structure at level +3.650, from [28].
Figure 4.5; Elevation drawing of the 1/4th scaled SMART2013-project structure, wall V01-V02, from [28].
Figure 4.6; Elevation drawing of the 1/4th scaled SMART2013-project structure, wall V03 from [28].
Figure 4.7: Elevation drawing of the 1/4th scaled SMART2013-project structure, wall V04, from [28].
4.3 Loads

To be able to see if the modeled structure has been properly modeled, its natural frequencies needs to be compared with the test model from SMART2013-project. For such a comparison, the loads that act vertically on the structure inclusive the self weight shall be the same as on the real structure. Here are the loads on the SMART2013-project defined.

4.3.1 Seismic action

The structure was tested according to French regulations with a PGA of 0.2g. The designed elastic spectrum applied corresponds to an earthquake of a 5.5 magnitude at a distance of 10 km [29]. The elastic response spectrum of the SMART2013-project specimen is shown in Figure 4.8.

![Figure 4.8: Elastic response spectra of the SMART2013-project specimen, from [29].](image)

4.3.2 Dead load

The dead load \( G \) of the SMART2013-project specimen is 2460 kg/m\(^3\) which includes both the concrete and the reinforcement. This value is similar to that applied in the FEM-Design software, which are 2500 kg/m\(^3\).

4.3.3 Live load

Figure 4.9 shows how additional loading is applied on the model. To be able to rescale the additional load on the specimen to a real structure scale, load values from the 1:4 scale model shall be multiplied by 16. The live load \( Q \) that is applied on the specimen and the model at each storey is presented in Table 4.1 [30].

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Figure 4.9; Additional applied load on the structure model, from [27].

Table 4.1; Live load on the specimen and the model at each storey

<table>
<thead>
<tr>
<th>Storey</th>
<th>Live load on 1:4 scale model [kN/m²]</th>
<th>Live load on modeled full scaled structure [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.600</td>
<td>73.6</td>
</tr>
<tr>
<td>2</td>
<td>5.225</td>
<td>83.6</td>
</tr>
<tr>
<td>3</td>
<td>5.200</td>
<td>83.2</td>
</tr>
</tbody>
</table>
4.4 Geometrical and material description

The total height of the real building is 14.6 m. Geometrical and material properties of beams, columns and walls are described in Table 4.2. Note that the dimensions are scaled to the real structure. On each story there are holes through the walls, smaller holes located on the shorter sides and larger holes located on the longer sides of the building. The geometrical properties of the holes are shown in Table 4.3, also with dimensions scaled to the real structure.

Table 4.2; Geometrical and material properties of beams, columns and walls

<table>
<thead>
<tr>
<th>Components</th>
<th>Length[m]</th>
<th>Thickness[m]</th>
<th>Height[m]</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall #V01+#V02</td>
<td>12.4</td>
<td>0.4</td>
<td>14.6</td>
<td>C30/37</td>
</tr>
<tr>
<td>Wall #V03</td>
<td>10.2</td>
<td>0.4</td>
<td>14.6</td>
<td>C30/37</td>
</tr>
<tr>
<td>Wall #V04</td>
<td>4.2</td>
<td>0.4</td>
<td>14.6</td>
<td>C30/37</td>
</tr>
<tr>
<td>Beam</td>
<td>5.8</td>
<td>0.6</td>
<td>1.3</td>
<td>C30/37</td>
</tr>
<tr>
<td>Column</td>
<td>14.6</td>
<td>0.8</td>
<td>0.8</td>
<td>C30/37</td>
</tr>
</tbody>
</table>

Table 4.3; Geometrical properties of holes through the walls

<table>
<thead>
<tr>
<th>Components</th>
<th>Width[m]</th>
<th>Height[m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smaller Holes</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Bigger Holes</td>
<td>5.0</td>
<td>3.6</td>
</tr>
</tbody>
</table>

4.5 Natural frequencies

To obtain the natural cyclic frequencies of the real structure from the tested model, the measured frequencies shall be multiplied by 0.5 [30]. The three first natural cyclic frequencies for the structure tested in the SMART2013-project, and the corresponding real structure, are presented in Table 4.4.

Table 4.4; Three first natural cyclic frequencies of the SMART2013-specimen

<table>
<thead>
<tr>
<th>No</th>
<th>Natural cyclic Frequency from 1:4 scaled model [Hz]</th>
<th>Corresponding natural cyclic frequency for the real structure [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.5</td>
<td>2.75</td>
</tr>
<tr>
<td>2</td>
<td>8.37</td>
<td>4.18</td>
</tr>
<tr>
<td>3</td>
<td>16.37</td>
<td>8.18</td>
</tr>
</tbody>
</table>
5. Seismic analysis
In this chapter the modeling procedure is presented shortly and thereafter loads that are defined in load combinations from Eurocode 8 and the DNB will be applied and analyzed separately. Natural modes of vibration, displacements and internal forces are illustrated later in this chapter.

5.1 Modeling
The modeling and analysis of the structure is done with the Swedish structural software FEM-Design 3D-structure version 12. The structure is assumed to be built on a hard rock ground, the most usual ground condition in Sweden where nuclear power plants are built. Therefore, all the bearing walls and columns have boundary condition fixed to the ground while the columns and beams have fixed-fixed boundary conditions. The structure is modeled according to Eurocode 8 and ASCE 4-98, the latter is also recommended in DNB [7], according which the Eurocode 8 is not suitable and applicable on nuclear power plants. However, the FEM-Design software for seismic analysis is based on Eurocode 8 from which some properties should be taken. The modeling was done in relation to the SMART2013-project, to be able to compare the natural cyclic frequencies from the modeled structure and the actual SMART2013-project model.

According to ASCE 4-98 (Table 3.1-1), the damping ratio for reinforced concrete structures are between 4-7%. The damping ratio chosen for the studied structure is 5%, the same as for the SMART2013-project. This value is also recommended by the FEM-Design software theory book [31] as well. In the calculation both torsional and second order effects are included to get the most critical value possible. The concrete strength properties were as in the SMART2013-project, which is C30/37. First the structure is modeled with the full scaled values in the FEM-Design 3D-structure, as given in Tables 4.2 and 4.3. Then two sets of loads and load combinations corresponding to each code (Eurocode 8 and DNB) are defined, and presented later in this chapter.

Next step is to determine the mesh size for the model. A proper mesh size makes the results accurate enough and the calculations time efficient. With smaller mesh size more accurate result will surely be obtained, but a reasonable accuracy with a proper calculation time interval is the goal. There is no recommendation in Eurocode 8 about mesh size, but according to ASCE 4-98 the size for the finite element model shall give results that are not significantly affected by further refinement in the element mesh size, to not risk the accuracy for getting more time efficient. Therefore a proper mesh size is here chosen to 0.16 m².

5.2 Seismic action
As was shown in section 2.1, the peak value of the ground acceleration, PGA can be read when the maximum value of \( f \) is considered. This can be seen in Figure 5.1 where the corresponding acceleration for a frequency of 100 Hz is amounted to 1.1 m/s². Therefore, the PGA value from the horizontal spectrum in Figure 5.1 is chosen to be 0.11g.
Figure 5.1: Horizontal envelope spectra; showing the PGA=0.11g with the maximum value of frequency, from [9].

5.3 Dead load
Based on concrete cylinders tested on the SMART2013-project, the density of the reinforced concrete was estimated to about $\rho_c = 2500 \text{ kg/m}^3$. The dead load $G$ of the structure is calculated as:

$$G = \rho_c \cdot g$$  \hspace{1cm} (5.1)

giving:

$$G = 2500 \cdot 10 = 25 \frac{\text{kN}}{\text{m}^3}$$

5.4 Live load
Eurocode has no specific recommendation for live load that should be used in nuclear facilities. Therefore, it should be estimated properly by the designer who should consider the load so that it includes the weight of the equipments that will be installed on the structure. The live load $Q$ is here given the same value as was applied on the SMART2013-project specimen. The 1st, 2nd and 3rd storeys are loaded with surface loads 20.8 kN/m$^2$, 21.5 kN/m$^2$ and 18.4 kN/m$^2$, respectively.
5.5 Natural frequencies

Natural frequencies of a structure are defined through the relation between stiffness and mass of the structure. Time required for an undamped system to complete one cycle of free vibration is called natural period of vibration $T_n$, defined as in Eq. (5.2). The natural circular frequency $\omega_n$, which is related to natural period of vibration, is given in Eq. (5.3) and the corresponding natural frequency of vibration $f_n$ in Eq. (5.4). The term “natural” used in $T_n$, $\omega_n$ and $f_n$ is used to emphasize that these properties are the natural properties of the system obtained through the free vibration without any external excitation.

\[
T_n = \frac{2\pi}{\omega_n} \quad (5.2)
\]

\[
\omega_n = \frac{k}{\sqrt{m}} \quad (5.3)
\]

\[
f_n = \frac{\omega_n}{2\pi} \quad (5.4)
\]

For a damped system with the damping ratio $\zeta$, Eq. (5.5) is valid for the natural frequency. Influence of damping ratio in both SDF (Single Degree of Freedom) and MDF (Multi Degree of Freedom) systems are small for ratios below 20%, damping often found for practical structures [32].

\[
\omega_{nd} = \omega_n \sqrt{1 - \zeta^2} \quad (5.5)
\]

5.6 Analysis according to Eurocode

5.6.1 Behavior factor

The behavior factor $q$, is calculated by the following equation:

\[
q = q_0 \cdot k_w \geq 1.5 \quad (5.6)
\]

The structure will be designed with a high ductility class DCH, due to the importance in being able to deform beyond the elastic range. Thus, for DCH, $q_0$ is selected from Table 3.2.

\[
q_0 = 4.0 \cdot \frac{\alpha_u}{\alpha_l} \quad (5.7)
\]
The ratio $\alpha_u/\alpha_1$ is amounted to 1.1; giving:

$$q_0 = 4.0 \cdot \frac{\alpha_u}{\alpha_1} = 4.0 \cdot 1.1 = 4.4$$

The system of the studied structure is not regular in elevation, therefore according to Eurocode 8, $q_0$ should be reduced by 20%:

$$q_0 = 4.4 \cdot (1 - 0.2) = 3.52$$

The factor $k_w$ is calculated as follows:

$$k_w = 0.5 \leq \frac{1 + \alpha_0}{3} \leq 1 \quad (5.8)$$

Where $\alpha_0$ is defined as:

$$\alpha_0 = \frac{\sum h_{wi}}{\sum l_{wi}} = \frac{h_{V1} + h_{V2} + h_{V3} + h_{V4}}{l_{V1} + l_{V2} + l_{V3} + l_{V4}} \quad (5.9)$$

giving:

$$\alpha_0 = \frac{14.4 + 14.4 + 14.4 + 14.4}{6.2 + 6.2 + 10.2 + 4.2} = 2.15$$

thus:

$$k_w = 0.5 \leq \frac{1 + \alpha_0}{3} \leq 1 = 0.5 \leq \frac{1 + 2.15}{3} \leq 1 = 0.5 \leq 1.05 \leq 1 = 1.00$$

With known values for $q_0$ and $k_w$, the behavior factor $q$ can be calculated:

$$q = q_0 \cdot k_w \geq 1.5$$

$$q = 3.52 \cdot 1.00 = 3.52 \geq 1.5 \text{ OK!}$$

The response spectra from SKI: report 1992 is modified by the behavior factor $q$ with accordance to Eurocode 8 and is illustrated in Figures 5.2 and 5.3 and corresponding Tables 5.1 and 5.2. The spectrums are reduced with factors which depend on the corresponding time interval on the spectra. In this case due to Swedish ground condition which is hard rock, the soil factor $S$ is equal to 1 [6]. Therefore the multiplicator of $a_g$ in Eqs. (3.7)-(3.10) are used as the factors applied as the modifier to SKI-reports spectrum, see Tables 5.1 and 5.2 and note that $T_B \approx T_c$. These reductions go up to 82% and are considered not suitable due to the magnification of the reduction. The spectrum from SKI-report is already designed for Swedish ground condition and since nuclear power plants in Sweden are in focus, it is not conservative to redesign the spectrum to obtain a smaller spectrum. It is mentioned in Eurocode 8 that the shape and values used in the spectrum may be found in National Annex for each country. Due to Swedish seismic conditions the structure is not considered passing the elastic deformations, therefore an elastic analysis is used in design and the behavior factor is not included.
Figure 5.2: SKI spectrum and corresponding modified horizontal spectrum according to Eurocode 8.

Table 5.1: Factors and corresponding modified $S_d$ according to Eurocode 8 for horizontal spectrum.

<table>
<thead>
<tr>
<th>$T$</th>
<th>$S_d$</th>
<th>factor</th>
<th>Eurocode 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0 \leq T \leq T_{B,C}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1,1</td>
<td>0,66</td>
<td>0,73</td>
</tr>
<tr>
<td>0,05</td>
<td>4</td>
<td>0,71</td>
<td>2,841</td>
</tr>
<tr>
<td>0,25</td>
<td>1,1</td>
<td>0,22</td>
<td>0,242</td>
</tr>
<tr>
<td>0,345</td>
<td>0,75</td>
<td>0,22</td>
<td>0,165</td>
</tr>
<tr>
<td>0,44</td>
<td>0,46</td>
<td>0,22</td>
<td>0,101</td>
</tr>
<tr>
<td>0,535</td>
<td>0,4</td>
<td>0,22</td>
<td>0,088</td>
</tr>
<tr>
<td>$T_{C} \leq T \leq T_{D}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0,63</td>
<td>0,3</td>
<td>0,22</td>
<td>0,066</td>
</tr>
<tr>
<td>0,725</td>
<td>0,2</td>
<td>0,22</td>
<td>0,044</td>
</tr>
<tr>
<td>0,82</td>
<td>0,175</td>
<td>0,22</td>
<td>0,038</td>
</tr>
<tr>
<td>0,915</td>
<td>0,15</td>
<td>0,22</td>
<td>0,033</td>
</tr>
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<td>1,01</td>
<td>0,14</td>
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<td>1,105</td>
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<td>0,028</td>
</tr>
<tr>
<td>1,2</td>
<td>0,12</td>
<td>0,22</td>
<td>0,026</td>
</tr>
<tr>
<td>$T_{D} \leq T$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1,274</td>
<td>0,11</td>
<td>0,22</td>
<td>0,024</td>
</tr>
<tr>
<td>1,347</td>
<td>0,1</td>
<td>0,22</td>
<td>0,022</td>
</tr>
<tr>
<td>1,421</td>
<td>0,095</td>
<td>0,22</td>
<td>0,021</td>
</tr>
<tr>
<td>1,495</td>
<td>0,09</td>
<td>0,22</td>
<td>0,02</td>
</tr>
<tr>
<td>1,568</td>
<td>0,085</td>
<td>0,22</td>
<td>0,019</td>
</tr>
<tr>
<td>1,642</td>
<td>0,08</td>
<td>0,22</td>
<td>0,018</td>
</tr>
<tr>
<td>1,716</td>
<td>0,075</td>
<td>0,22</td>
<td>0,016</td>
</tr>
<tr>
<td>1,789</td>
<td>0,07</td>
<td>0,22</td>
<td>0,015</td>
</tr>
<tr>
<td>1,863</td>
<td>0,065</td>
<td>0,22</td>
<td>0,014</td>
</tr>
<tr>
<td>1,936</td>
<td>0,06</td>
<td>0,22</td>
<td>0,013</td>
</tr>
</tbody>
</table>
Figure 5.3: SKI spectrum and corresponding modified vertical spectrum according to Eurocode 8.

Table 5.2: Factors and corresponding modified $S_d$ according to Eurocode 8 for vertical spectrum.

<table>
<thead>
<tr>
<th>Vertical spectrum from SKI: report 1992</th>
<th>Eurocode 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_d$</td>
<td>$T$</td>
</tr>
<tr>
<td>----------------------------------------</td>
<td>------------</td>
</tr>
<tr>
<td>0,9</td>
<td>0</td>
</tr>
<tr>
<td>3,5</td>
<td>0,05</td>
</tr>
<tr>
<td>1</td>
<td>0,15</td>
</tr>
<tr>
<td>0,5</td>
<td>0,235</td>
</tr>
<tr>
<td>0,35</td>
<td>0,32</td>
</tr>
<tr>
<td>0,25</td>
<td>0,405</td>
</tr>
<tr>
<td>0,18</td>
<td>0,49</td>
</tr>
<tr>
<td>0,15</td>
<td>0,575</td>
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<tr>
<td>0,1</td>
<td>0,66</td>
</tr>
<tr>
<td>0,09</td>
<td>0,745</td>
</tr>
<tr>
<td>0,08</td>
<td>0,83</td>
</tr>
<tr>
<td>0,075</td>
<td>0,915</td>
</tr>
<tr>
<td>0,07</td>
<td>1</td>
</tr>
<tr>
<td>0,065</td>
<td>1,037</td>
</tr>
<tr>
<td>0,06</td>
<td>1,074</td>
</tr>
<tr>
<td>0,055</td>
<td>1,111</td>
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<tr>
<td>0,052</td>
<td>1,148</td>
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<tr>
<td>0,05</td>
<td>1,185</td>
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<td>0,048</td>
<td>1,222</td>
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<tr>
<td>0,046</td>
<td>1,259</td>
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<tr>
<td>0,044</td>
<td>1,295</td>
</tr>
<tr>
<td>0,042</td>
<td>1,332</td>
</tr>
<tr>
<td>0,04</td>
<td>1,369</td>
</tr>
</tbody>
</table>
5.6.2 Load combination for vertical actions
The following load combination is valid:

\[ \sum G_{k,j} + \sum \psi_{E,i} \cdot Q_{k,i} \]  \hspace{1cm} (5.10)

where \( \psi_{E,i} = \varphi \cdot \psi_{2,i} = 1.0 \cdot 0.9 = 0.9 \), see section 2.2.2. From Eq. (5.10) the load combination for vertical actions can be expressed as:

\[ \sum G_{k,j} + 0.9 \cdot \sum Q_{k,i} \]  \hspace{1cm} (5.11)

5.6.3 Load combination for seismic design situation
The following load combination for seismic design is valid:

\[ \sum_{j \geq 1} G_{k,j} + P + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} Q_{k,i} \]  \hspace{1cm} (5.12)

By insertion of \( A_{Ed} = \gamma_1 \cdot A_{Ek} \), the following expression is written:

\[ \sum_{j \geq 1} G_{k,j} + P + \gamma_1 \cdot A_{Ek} + \sum_{i \geq 1} \psi_{2,i} Q_{k,i} \]  \hspace{1cm} (5.13)

where \( \gamma_1=1.4 \), see Table 2.4 and \( P = 0.0 \) due to no prestressed reinforcements. Therefore, the following combination is used:

\[ \sum_{j \geq 1} G_{k,j} + 1.4 \cdot A_{Ek} + 0.9 \cdot \sum_{i \geq 1} Q_{k,i} \]  \hspace{1cm} (5.14)
5.6.4 Modal response spectrum analysis

According to what FEM-Design theory book [31], Eurocode 8 and Norwegian Standards indicate that sum of the effective mass of the chosen mode shapes – at least horizontal direction – should reach 90% of total mass. Additionally every mode shape has to be taken into account when effective mass is larger than 5%. Three fundamental vibration modes of the structure are given in Table 5.3 where \( T \) is the period of the vibration and \( m_x, m_y \) and \( m_z \) represent the effective mass moments in \( x, y \) and \( z \) direction. The presented natural vibration modes corresponding to the structures natural cyclic frequency is for a system with a damping ratio of 5%. Figures 5.4 to 5.15 show the first three natural vibrations for corresponding natural frequencies of 2.37 Hz, 4.20 Hz and 8.13 Hz, respectively. Figures 5.16 to 5.19 show the displacement of the structure at corners A-D shown on the model, due to the designing load combination from Eurocode 8. The maximum displacement of the structure is also shown which is titled to E. According the conditions defined in FEM-Design theory book, the number of chosen mode shapes for the analysis is 15 where first three modes of the structure are included. These shapes are presented in Table 5.4. For combination of modal responses Eurocode 8 defines conditions which are here presented in Eq. (3.14) therefore Eq. (5.15) is applied. The condition is not satisfied, and for more accurate procedure of combining the modal responses, CQC method shall be used.

### Table 5.3: Three fundamental vibration modes of the structure

<table>
<thead>
<tr>
<th>No</th>
<th>( T ) [s]</th>
<th>( m_x ) [%]</th>
<th>( m_y ) [%]</th>
<th>( m_z ) [%]</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.422</td>
<td>63.8</td>
<td>1.0</td>
<td>0.0</td>
<td>Swaying in ( x ) and ( y ) direction</td>
</tr>
<tr>
<td>2</td>
<td>0.238</td>
<td>0.0</td>
<td>72.7</td>
<td>0.0</td>
<td>Swaying in ( y ) direction</td>
</tr>
<tr>
<td>3</td>
<td>0.123</td>
<td>16.7</td>
<td>0.0</td>
<td>8.8</td>
<td>Swaying in ( x ) and ( z ) direction</td>
</tr>
</tbody>
</table>

\[
T_j \leq 0.9 \cdot T_i \quad ; \quad T_j \leq T_i \quad \text{(5.15)}
\]

\[
T_2 \leq T_1 \cdot 0.9 \rightarrow 0.238 \leq 0.422 \cdot 0.9 \rightarrow 0.238 \leq 0.3798 \quad \text{SATISFFIED!}
\]

\[
T_3 \leq T_2 \cdot 0.9 \rightarrow 0.123 \leq 0.238 \cdot 0.9 \rightarrow 0.123 \leq 0.2142 \quad \text{SATISFFIED!}
\]

\[
T_4 \leq T_3 \cdot 0.9 \rightarrow 0.118 \leq 0.123 \cdot 0.9 \rightarrow 0.118 \leq 0.1107 \quad \text{SATISFFIED!}
\]

\[
T_5 \leq T_4 \cdot 0.9 \rightarrow 0.116 \leq 0.118 \cdot 0.9 \rightarrow 0.116 \leq 0.1062 \quad \text{NOT SATISFFIED!}
\]
Table 5.4: Mode shapes chosen for analysis

<table>
<thead>
<tr>
<th>No</th>
<th>( T[s] )</th>
<th>( m_x[%] )</th>
<th>( m_y[%] )</th>
<th>( m_z[%] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.422</td>
<td>63.8</td>
<td>1.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>0.238</td>
<td>0.0</td>
<td>72.7</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>0.123</td>
<td>16.7</td>
<td>0.0</td>
<td>8.8</td>
</tr>
<tr>
<td>4</td>
<td>0.118</td>
<td>5.2</td>
<td>1.2</td>
<td>40.8</td>
</tr>
<tr>
<td>5</td>
<td>0.116</td>
<td>0.0</td>
<td>0.0</td>
<td>10.2</td>
</tr>
<tr>
<td>6</td>
<td>0.113</td>
<td>0.5</td>
<td>1.5</td>
<td>0.8</td>
</tr>
<tr>
<td>7</td>
<td>0.106</td>
<td>1.7</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>8</td>
<td>0.105</td>
<td>4.2</td>
<td>0.0</td>
<td>0.7</td>
</tr>
<tr>
<td>9</td>
<td>0.098</td>
<td>0.0</td>
<td>0.0</td>
<td>5.2</td>
</tr>
<tr>
<td>10</td>
<td>0.079</td>
<td>0.0</td>
<td>5.4</td>
<td>2.6</td>
</tr>
<tr>
<td>11</td>
<td>0.078</td>
<td>0.0</td>
<td>4.4</td>
<td>0.0</td>
</tr>
<tr>
<td>12</td>
<td>0.076</td>
<td>0.0</td>
<td>6.8</td>
<td>0.8</td>
</tr>
<tr>
<td>13</td>
<td>0.070</td>
<td>3.1</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>14</td>
<td>0.056</td>
<td>0.0</td>
<td>1.2</td>
<td>0.6</td>
</tr>
<tr>
<td>15</td>
<td>0.032</td>
<td>0.0</td>
<td>0.0</td>
<td>11.5</td>
</tr>
</tbody>
</table>

\[ \sum m_{eff} \quad 95.1 \quad 94.3 \quad 82.1 \]
Figure 5.4; Natural vibration mode 1, 2.37 Hz - Side view

Figure 5.5; Natural vibration mode 1, 2.37 Hz - Side view
Figure 5.6; Natural vibration mode 1, 2.37 Hz - Top view

Figure 5.7; Natural vibration mode 1, 2.37 Hz - 3D view
Figure 5.8; Natural vibration mode 2, 4.20 Hz - Side view

Figure 5.9; Natural vibration mode 2, 4.20 Hz - Side view
Figure 5.10; Natural vibration mode 2, 4.20 Hz - Top view

Figure 5.11; Natural vibration mode 2, 4.20 Hz - 3D view
Figure 5.12; Natural vibration mode 3, 8.13 Hz - Side view

Figure 5.13; Natural vibration mode 3, 8.13 Hz - Side view
Figure 5.14; Natural vibration mode 3, 8.13 Hz - Top view

Figure 5.15; Natural vibration mode 3, 8.13 Hz - 3D view
Figure 5.16; Maximum displacement due to Eurocode 8 load combination in mm, side view

Figure 5.17; Maximum displacement due to Eurocode 8 load combination in mm, side view
Figure 5.18; Maximum displacement due to Eurocode 8 load combination in mm, top view

Figure 5.19; Maximum displacement due to Eurocode 8 load combination in mm, 3D view
Resulting $M_y$ from Eurocode 8 load combination on the primary seismic beam at top level by superposition from acting loads is presented in Figure 5.20 to illustrate how the resulting moment is obtained from the load combination.

\[
M_y \text{ from dead load} \times 1.0
\]

\[
M_y \text{ from live load} \times 0.9
\]

\[
M_y \text{ from seismic load} \times 1.4
\]

\[
M_y \text{ from load combination}
\]

Figure 5.20: Total $M_y$ of the primary seismic beam at top level by superposition of acting loads
5.7 Analysis according to DNB

5.7.1 Load combination for seismic design situation
Water pressure $H_{gw}$, earth pressure $H_{ge}$, water pressure difference between normal water level and time variable water level $H_{qw}$ and Soil pressure due to movable surface load $H_{qe}$ are chosen to be 0 due to the ground condition of the studied structure that is assumed to be found on hard rock and over the earths surface. The shrinkage $e_{cs}$, settlement $\delta_s$ and climate related temperature load $\Delta T$ are assumed being neglectable. Process related loads during normal operation and shutdown period $M_n$ and process related loads during operation disturbance $M_d$ is a very expansive and time taking process to define. These loads are considered small compared to other types of loads such as dead weight $D_k$, live load $L$, snow load $S$, wind load $W_q$ and load due to designed DBE, $E_{DBE}$. Therefore $M_n$ and $M_d$ are considered neglectable. With a value of $\psi_2 = 0.9$ for live load, also taken from section 2.2.2, Table 2.3, $\psi_2 = 0.2$ for snow load and $\psi_2 = 0$ for wind load, the following load combination with the assumptions above gives:

$$1.0D_k + 1.0 \cdot \psi_2 S + 1.0 \cdot \psi_2 L + 1.0E_{DBE}$$ (5.16)

$$1.0D_k + 0.2 \cdot S + 0.9 \cdot L + 1.0E_{DBE}$$ (5.17)

Dead load and live load are presented in sections 5.3 and 5.4. The design value for the snow load is calculated as:

$$S = \mu_i C_e C_t s_k$$ (5.18)

where $\mu_i$ is the snow load shape coefficient, $C_e$ is the exposure coefficient, $C_t$ is the thermal coefficient, $s_k$ is the characteristic value of snow load. To determine the characteristic value of snow load a region must be selected for the analysis. Here the region is 60 km south of Gothenburg close to where the Ringhals nuclear power plant is located. According to Eurocode 1 $s_k, C_t, C_e, \mu_i$ are respectively amounted to 2 kN/m$^2$, 1.0, 1.2 and 0.8, respectively [11].

5.7.2 Modal response spectrum analysis
The three fundamental mode shapes obtained are presented in Table 5.5. According the conditions presented in section 5.6.4, the number of chosen mode shapes for the analysis is 15. These modes are presented in Table 5.6. The first three natural vibration modes and displacements are also shown in Figures 5.21-5.36.
Table 5.21: Three fundamental vibration modes of the structure

<table>
<thead>
<tr>
<th>No</th>
<th>(T) [s]</th>
<th>(m_x) [%]</th>
<th>(m_y) [%]</th>
<th>(m_z) [%]</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.425</td>
<td>63.9</td>
<td>1.0</td>
<td>0.0</td>
<td>Swaying in x and y direction</td>
</tr>
<tr>
<td>2</td>
<td>0.239</td>
<td>0.0</td>
<td>72.7</td>
<td>0.0</td>
<td>Swaying in y direction</td>
</tr>
<tr>
<td>3</td>
<td>0.124</td>
<td>15.6</td>
<td>0.0</td>
<td>9.4</td>
<td>Swaying in x and z direction</td>
</tr>
</tbody>
</table>

Table 5.22: Mode shapes chosen for analysis

<table>
<thead>
<tr>
<th>No</th>
<th>(T) [s]</th>
<th>(m_x) [%]</th>
<th>(m_y) [%]</th>
<th>(m_z) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.425</td>
<td>63.9</td>
<td>1.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>0.239</td>
<td>0.0</td>
<td>72.7</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>0.124</td>
<td>15.6</td>
<td>0.0</td>
<td>9.4</td>
</tr>
<tr>
<td>4</td>
<td>0.119</td>
<td>6.1</td>
<td>1.3</td>
<td>39.5</td>
</tr>
<tr>
<td>5</td>
<td>0.117</td>
<td>0.0</td>
<td>0.0</td>
<td>10.2</td>
</tr>
<tr>
<td>6</td>
<td>0.113</td>
<td>0.0</td>
<td>1.6</td>
<td>1.0</td>
</tr>
<tr>
<td>7</td>
<td>0.106</td>
<td>2.5</td>
<td>0.0</td>
<td>0.6</td>
</tr>
<tr>
<td>8</td>
<td>0.105</td>
<td>3.5</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>9</td>
<td>0.098</td>
<td>0.0</td>
<td>0.0</td>
<td>5.5</td>
</tr>
<tr>
<td>10</td>
<td>0.080</td>
<td>0.0</td>
<td>3.9</td>
<td>2.0</td>
</tr>
<tr>
<td>11</td>
<td>0.079</td>
<td>0.0</td>
<td>6.4</td>
<td>0.0</td>
</tr>
<tr>
<td>12</td>
<td>0.076</td>
<td>0.0</td>
<td>6.1</td>
<td>0.9</td>
</tr>
<tr>
<td>13</td>
<td>0.070</td>
<td>3.1</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>14</td>
<td>0.056</td>
<td>0.0</td>
<td>1.1</td>
<td>0.6</td>
</tr>
<tr>
<td>15</td>
<td>0.032</td>
<td>0.0</td>
<td>0.0</td>
<td>11.5</td>
</tr>
</tbody>
</table>

\[ \sum m_{\text{eff}} \]

94.6 93.9 81.2
Figure 5.21; Natural vibration mode 1, 2.35 Hz - Side view

Figure 5.22; Natural vibration mode 1, 2.35 Hz - Side view
Figure 5.23; Natural vibration mode 1, 2.35 Hz - Top view

Figure 5.24; Natural vibration mode 1, 2.35 Hz - 3D view
Figure 5.25; Natural vibration mode 2, 4.18 Hz - Side view

Figure 5.26; Natural vibration mode 2, 4.18 Hz - Side view
Figure 5.27; Natural vibration mode 2, 4.18 Hz - Top view

Figure 5.28; Natural vibration mode 2, 4.18 Hz - 3D view
Figure 5.29; Natural vibration mode 3, 8.10 Hz - Side view

Figure 5.30; Natural vibration mode 2, 4.18 Hz - Side view
Figure 5.31: Natural vibration mode 2, 4.18 Hz - Top view

Figure 5.32: Natural vibration mode 2, 4.18 Hz - 3D view
Figure 5.33; Maximum displacement due to DNB load combination in mm, side view

Figure 5.34; Maximum displacement due to DNB load combination in mm, side view
Figure 5.35; Maximum displacement due to DNB load combination in mm, top view

Figure 5.36; Maximum displacement due to DNB load combination in mm, 3D view
Resulting $M_y$ from DNB load combination on the primary seismic beam at top level by super position from acting loads is presented in Figure 5.37 to illustrate how the resulting moment is obtained from the load combination.

\[ M_y \text{ from dead load} \times 1.0 + M_y \text{ from live load} \times 0.9 + M_y \text{ from snow load} \times 0.2 + M_y \text{ from seismic load} \times 1.0 = \]

Figure 5.37; Total $M_y$ of the primary seismic beam at top level by super position of acting loads
6. Example according to Eurocode 8

In this chapter the methods described in chapters 2 and 3 will be applied on the studied structure. In this thesis the design of the primary seismic beams and columns is in focus. Therefore the beam with the largest bending moment and shear force will be designed as an example of design according to Eurocode 8. The same calculation will be used to determine the lowest acceptable bending resistance of the column subjected to the largest bending moment. Note that the calculations here are based on Eurocode 8 only. This means that a new loading conditions are presented and applied on the studied model. Therefore, Eq. (5.14) is valid for this example, with a live load assumed to be 25 kN/m². The response spectrum used is the Swedish spectrum presented in section 2.1.

6.1 Structural Regularity

The regularity and irregularity of the structure is defined in plan and elevation and the type of the analysis that should be done for the structure is dependent on it.

6.1.1 Regularity in plan

The conditions for regularity in plan, according to Eurocode 8 are shown below.

- \( \lambda = \frac{l_{\text{max}}}{l_{\text{min}}} \leq 4 \)
- \( e_{0(x,y)} \leq 0.30 \cdot r_{(x,y)} \)
- \( r_{(x,y)} \geq l_s \)

where:

\[ \lambda = \frac{l_{\text{max}}}{l_{\text{min}}} \]  

(6.1)

giving:

\[ \lambda = \frac{12.4}{4.2} = 2.95 \leq 4 \]

According to simplification for multi storey buildings in Eurocode 8, this building can be satisfied as regular in plan.

6.1.2 Regularity in Elevation

The condition that should be fulfilled in the studied case is:

\[ \frac{L - L_2}{L} \leq 0.30 \]  

(6.2)

here is:

\[ L = 10.2 \text{ m} \text{ and } L_2 = 4.2 \text{ m} \]

which gives:

\[ 0.59 \leq 0.30; \text{ NOT fulfilled!} \]
This buildings regularity conditions are not fulfilled and it is categorized as not regular in elevation and regular in plan. Therefore, according Eurocode 8, linear elastic analysis of this building shall be done with modal analysis and its behavior factor for linear analysis shall be a decreased value.

6.2 Structural type of the building
The resultant of shear forces at the building base, acting on each structural wall and column, is calculated by the FEM-Design software and it is presented below:

**Table 6.1: Resultant of shear forces at the building base.**

<table>
<thead>
<tr>
<th>Component</th>
<th>Shear resistance [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall#01</td>
<td>5126.7</td>
</tr>
<tr>
<td>Wall#02</td>
<td>4806.1</td>
</tr>
<tr>
<td>Wall#03</td>
<td>4365.9+4746.9=9112.8</td>
</tr>
<tr>
<td>Wall#04</td>
<td>6872.4</td>
</tr>
<tr>
<td>Column</td>
<td>3834.9</td>
</tr>
</tbody>
</table>

Total shear resistance of the building, \( V_{\text{tot}} \) is here:

\[
V_{\text{tot}} = V_{\text{Wall},V01} + V_{\text{Wall},V02} + V_{\text{Wall},V03} + V_{\text{Wall},V04} + V_{\text{Column}} = 29753 \text{ kN}
\]

The total wall shear resistance of the building, \( \sum V_{\text{Wall},i} \) is amounted to:

\[
\sum V_{\text{Wall},V,i} = 25918.047 \text{ kN}
\]

where \( V_{\text{Wall},V,i} \) is the shear resistance of each structural wall \( i \). Thus:

\[
\frac{\sum V_{\text{Wall},V,i}}{V_{\text{tot}}} = \frac{25918.047}{29753.043} = 0.87 = 87\%
\]  \hspace{1cm} (6.3)

Structural walls will take more than 65% of the total shear resistance of the whole structural system. Therefore the structural type of the building is chosen to “wall system”.

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6.3 Design for DCH
Analysis of the structure with the designing seismic load combination was done using the FEM-Design software. The primary seismic beam on the top level of the building is designed and calculated since it is subjected to the largest moment and shear force compared to other primary seismic beams of the structure.

6.3.1 Material requirements
The concrete strength class of this structure is chosen to be C30/37, this is the same strength class that was used in the SMART2013-project. Reinforcement bars are chosen from class C according to Eurocode 8. Due to that this class reinforcements have higher ductile capacity than type A and B. The reinforcement bars has the characteristic yield strength \( f_{yk} = 500 \text{ MPa} \) with total elongation at maximum force that is \( \varepsilon_u = 7.5\% \).

6.3.2 Beams
The studied structure has one beam at each level that is supported by the column and shear wall V01+V02 with the distance \( l_{0t} = 6.40 \text{ m} \). The width of the beam is chosen to be as the beam width in the SMART2013-project, 600 mm which is larger than 200 mm that is the requirement. The width to height ratio of the beam should satisfy the following expressions:

\[
\frac{l_{0t}}{b} \leq \frac{70}{\left(\frac{h}{b}\right)^{3/4}} \tag{6.4}
\]

\[
h/b \leq 3.5 \tag{6.5}
\]

From Eq. (6.4) the following is obtained:

\[
\frac{6.40}{1.3} \leq \frac{70}{\left(\frac{1.3}{0.6}\right)^{1/3}} \rightarrow 4.92 \leq 54.1 \quad \text{OK!}
\]

and from Eq. (6.5):

\[
\frac{1.3}{0.6} \leq 3.5 \rightarrow 2.17 \leq 3.5 \quad \text{OK!}
\]

According to Eurocode 8 the width of the beam should satisfy the following expression as well:

\[
b_w \leq \min\{b_c + h_w; 2b_c\} \tag{6.6}
\]
giving:

\[
0.6 \leq \min\{0.8 + 1.3; 2 \cdot 0.8\} = 0.6 \leq 1.6 \quad \text{OK!}
\]
6.3.3 Columns and ductile walls
For ductile walls Eurocode 8 defines an expression for the web thickness $b_{wo}$, which is 400 mm in the studied case:

$$b_{wo} \geq \max\{0.15, \frac{h_s}{20}\} \quad (6.7)$$

thus:

$$0.4 \geq \max\{0.15, \frac{4.8}{20}\}$$

$$0.4 \geq 0.24 \quad \text{OK!}$$

6.3.4 Design for shear resistance
The shear forces where obtained by the analysis of primary seismic beams and is illustrated in Figure 6.1. For designing the beam, first it should be known that if the beam can carry the shear forces or not, which can be calculated by Eq. (6.8). The beam itself without shear reinforcement can carry a shear force of 221 kN. By looking closer at the beam, the regions of the beam subjected to a larger shear force than 221 kN can be found. This was done by finding the nearest calculated shear force in the FEM-Design analysis. The distance needed to be shear reinforced is taken from the nearest shear forces compared with $V_{Rd,c}$, as shown in Figure 6.2. As earlier mentioned, the calculation will be on the conservative side due to the structures importance and lack of recommendations regarding seismic design of nuclear power plants in the Eurocode 8. Therefore the designing shear force will be the maximum value in each section.

![Figure 6.1; Shear force acting on the primary seismic beams. Values are given in kN.](image-url)
\[ V_{Rd,c} = \left( \frac{0.18}{\gamma_c} \cdot k \cdot \frac{3}{\sqrt{100 \rho_1 \cdot f_{ck} + 0.15 \sigma_{cp}}} \right) \cdot b_w d \]  

\[ \gamma_c = 1.5 \]

\[ k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \rightarrow 1 + \sqrt{\frac{200}{1300}} \leq 2.0 \rightarrow 1.4 \leq 2.0 \]

\[ \rho_1 = \frac{A_{sl}}{b_w d} \leq 0.02 \rightarrow \frac{6 \cdot \left( \frac{0.016^2 \cdot \pi}{4} \right)}{0.6 \cdot 1.250} = 0.0016 \leq 0.02 \]

\[ f_{ck} = 30 \text{ MPa} \]

\[ N_{Ed} = 0 \rightarrow \sigma_{cp} = \frac{N_{Ed}}{A_c} = 0 \]

\[ b_w = 0.6 \text{ m} \]

\[ d = 1.3 \text{ m} \]

thus:

\[ V_{Rd,c} = \left( \frac{0.18}{1.5} \cdot 1.4 \cdot \sqrt[3]{100 \cdot 0.0016 \cdot 30 + 0.15 \cdot 0} \right) \cdot 0.6 \cdot 1.3 = 221 \text{ kN} \]

*Figure 6.2; Distances that is needed to be shear reinforced is presented as section A and B.*
Reinforcement needed for section A

The reinforcement needed to be applied is calculated as:

$$\frac{A_{sw}}{s} = \frac{V_{LA}}{f_{ywd} \cdot z \cdot (\cot \theta + \cot \alpha \sin \alpha)} \quad (6.9)$$

where $V_{LA} = 428.8 - 207 = 221.8 \text{ kN}$. According to the Eurocode the compression strut inclination $\theta$ must be set equal to $45^\circ$. The shear reinforcement is chosen to be vertical $\alpha = 90^\circ$, which is common in practice. Therefore:

$$\begin{align*}
\cot \theta &= 1 \\
\cot \alpha &= 0 \\
\sin \alpha &= 1 \\
n &= 0.9 \cdot d = 1.125 \text{ m} \\
f_{ywd} &= 435 \text{ MPa}
\end{align*}$$

will give:

$$\frac{A_{sw}}{s} = \frac{221.8}{435 \cdot 10^3 \cdot 1.125 \cdot (1 + 0) \cdot 1} = 453.23 \frac{\text{mm}^2}{\text{m}}$$

By using reinforcement bars with 10 mm diameters the following spacing is obtained:

$$s = \frac{A_{sw}}{453.23} = \frac{2 \cdot \pi \cdot 10^2}{4 \cdot 453.23} = 0.346 \text{ m} \rightarrow s = 300 \text{ mm}$$

A suitable reinforcement in section A, for $453.23 \frac{\text{mm}^2}{\text{m}}$, is $\varnothing 10 \times 300$.

Reinforcement needed for section B

The reinforcement needed is calculated as it was calculated in section A:

$$\frac{A_{sw}}{s} = \frac{V_{LB}}{f_{ywd} \cdot z \cdot (\cot \theta + \cot \alpha \sin \alpha)} \quad (6.10)$$
where $V_{1A} = 969.4 - 218 = 751$ kN. Therefore:

$$A_{sw} = \frac{751}{435 \cdot 10^3 \cdot 1.125 \cdot (1 + 0) \cdot 1} = 1535 \text{ mm}^2 / \text{m}$$

By using reinforcement bars with 10 mm in diameters the following spacing is obtained:

$$S = \frac{A_{sw}}{1535} = \frac{2 \cdot \pi \cdot 10^2}{4 \cdot 1535} = 0.102 \text{ m} \rightarrow S = 100 \text{ mm}$$

The suitable reinforcement in section B, for $1535 \text{ mm}^2 / \text{m}$, is here $\Phi 10 \times 100$. 

**Critical regions of the beam**

The critical regions of the beam are taken from the end cross-sections into the beam with length $l_{cr}$.

$$l_{cr} = 1.5 h_w = 1.5 \cdot 1.3 = 1.95 \text{ m} \approx 2 \text{ m}$$

According to Eurocode 8 the spacing of the shear reinforcement bars shall satisfy:

$$s = \min \left\{ \frac{h_w}{4}; 24d_{bw}; 175; 6d_{bl} \right\}$$

(6.11)

giving:

$$s = \min \left\{ \frac{1.3}{4}; 24 \cdot 10; 175; 6 \cdot 16 \right\} = 96 \text{ mm} \rightarrow S = 75 \text{ mm}$$

**6.3.5 Design for bending resistance**

For design of the beam bending resistance, the bending moment due to the maximum load combination is considered. The bending moments obtained by the analysis on primary seismic beams are illustrated in Figure 6.3. The maximum moment acting on the beam can be seen in Figure 6.4. At first height of the compression zone is calculated as is shown in Eq. (6.12).
Figure 6.3: Moment diagram on the primary seismic beams. Values are given in kNm.

\[
\frac{x}{d} = \frac{1}{2\beta} \left[ 1 \sqrt{\frac{1}{4\beta^2} \left( \frac{M}{\alpha \beta f_{cd} bd^2} \right)} \right] \\
\beta = 0.45 \\
\alpha = 0.8 \\
b = 0.6 \text{ m} \\
d = 1.250 \text{ m} \\
f_{cd} = \frac{30}{1.5} = 20 \text{ MPa} \\
M = 634 \text{ kNm}
\]

Thereafter the following is valid:

\[
x = \frac{1}{2\beta} \left[ 1 \sqrt{\frac{1}{4\beta^2} \left( \frac{M}{\alpha \beta f_{cd} bd^2} \right)} \right] \cdot d \\
\] (6.13)
Figure 6.4: Moment distribution along the studied beam. Max moment is 634 kNm.

By knowing the compression zone height, the required reinforcement can be calculated, as:

\[
x = \left[ \frac{1}{2 \cdot 0.45} - \frac{1}{4(0.45)^2} - \frac{634}{0.8 \cdot 0.45 \cdot 20 \cdot 0.6 \cdot 1.250^2} \right] \cdot 1.250 = 52\text{mm}
\]

\[
x_{\text{cd}} = \frac{\alpha f_{cd} b x}{f_{yd}}
\]

By choosing reinforcement bars with diameter 16 mm, the suitable amount of reinforcement is 6 × Ø16. Anchorage length of the reinforcement is calculated as:

\[
l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,\text{reqd}} \geq l_{b,\text{min}}
\]

Where the coefficients are calculated as follows, in accordance with Eurocode 2:

\[
\alpha_1 = 1.0; \text{ due to straight bars.}
\]

\[
\alpha_2 = 0.7 \leq 1 - 0.15 \left( \frac{c_d}{\varphi} - 1 \right) \leq 1.0; \text{ } c_d = \min \left( \frac{a}{\varphi}, c_1, c \right) = 40 \text{mm} \rightarrow \alpha_2 = 0.775
\]
\[ \alpha_3 = 0.7 \leq 1 - K\lambda \leq 1.0; K = 0.1; \quad \lambda = (\Sigma A_{st} - \Sigma A_{st,min})/A_s \]

The minimum anchorage length \( l_{b,min} \) is used instead with the requirement that it should be smaller than the designed anchorage length to be on the safe side. Here is:

\[
l_{b,min} = \max(0.3 l_{b,rqd}, 100, 100 \text{ mm}) \quad \text{where: } l_{b,rqd} = \frac{\varnothing}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} = \frac{16}{4} \cdot \frac{435}{f_{bd}}; \\
f_{bd} = 2.25 \cdot \eta_1 \eta_2 \cdot f_{ctd} \quad \text{where: } f_{ctd} = 1.4, \eta_1 = 1.0 \text{ and } \eta_2 = 1.0 \text{ giving: } f_{bd} = 3.15 \text{ MPa}; \\
\]

will give: \( l_{b,rqd} = 552.4 \text{ mm} \rightarrow l_{b,min} = \max(166, 160, 100) = 166 \text{ mm} \)

Number of transverse bars within the minimum anchorage length is \( 166/75 = 2 \) bars. 75 mm is the distance between the shear reinforcements in the critical region of the beam. Therefore value for \( \lambda \) is calculated to:

\[
\lambda = \frac{\left( 0.01^2 \cdot \pi \right) - \left( 0.016^2 \cdot \pi \right)}{0.016^2} = 0.25 \rightarrow \alpha_3 = 0.95 \\
\alpha_{4,5} = 1 \\
\alpha_2 \cdot \alpha_3 \cdot \alpha_4 = 0.74 \geq 0.7 \quad \text{OK!} 
\]

The designed anchorage length is now calculated to:

\[
l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,min} \\
l_{bd} = 1 \cdot 0.775 \cdot 0.95 \cdot 1 \cdot 1 \cdot 552.4 \geq 166 \\
l_{bd} = 407 \text{ mm} \geq 166 \text{ mm} \quad \text{OK!} 
\]

**Top reinforcement:**

Calculated top reinforcement indicates that for a distance of 2.13 m from the column-beam joint, \( 2 \times \varnothing 16 \) should be provided. Also a distance of 1.76 m from the wall-beam joint shall be provided with \( 1 \times \varnothing 16 \). As mentioned, at least two bars with diameter of 14 mm shall be provided along the length of the beam, both at top and bottom. Therefore, the suitable amount of top reinforcement is \( 2 \times \varnothing 16 \) along the entire length of the beam.
**Bottom reinforcement:**
Calculated bottom reinforcement with two layers of reinforcement gives one layer of reinforcement with $7 \times \varnothing 16 - 6700$ mm and another layer with $3 \times \varnothing 16 - 8500$ mm. In this case the two bars with diameter 14 mm, that should run along the entire beam length is included in the reinforcement group with $3 \times \varnothing 16 - 8500$ mm.

By knowing the designed value for bending resistance in the primary seismic beam, the joining column must at least have 30% higher bending resistance than the beam to prevent the creation of plastic hinges, see section 3.7.4. In this case the minimum bending resistance of the column is calculated as 842.2 kNm:

\[
\sum M_{Re} \geq 1.3 \sum M_{Rb} \quad (6.16)
\]

\[
\sum M_{Re} \geq 1.3 \cdot 634 \text{ kNm}
\]

\[
\sum M_{Re} \geq 824.2 \text{ kNm}
\]
7. Discussion and comparison of results

The purpose of this thesis is to evaluate the difference between seismic design of nuclear concrete structures according to Eurocode 8 and DNB, from a purely structural point of view and therefore the safety in design of such an important structure plays a vital role. As mentioned earlier Eurocode clearly cites that special structures such as nuclear power plants are beyond its scope. DNB also cites that Eurocode 8 is not applicable for nuclear power plants, therefore DNBs instructions for seismic design is taken from ASCE 4-98 [7]. The studied structure is modeled in FEM-Design software and loaded according to the guidelines from Eurocode 8 and the DNB. First the natural frequencies obtained from the SMART2013-project specimen were compared with the modeled computerized structure to see if the structure is modeled in a proper way. Thereafter, results from Eurocode and DNB are compared. As mentioned, natural circular frequency depend on mass and stiffness of the structure. From Eqs. (5.3)-(5.5) it is clear that increasing mass will result in a lower natural circular frequency which in turn gives a lower natural cyclic frequency. With mass of the structure is meant the vertical loads that act on the structure, including the gravity load. According to Eqs. (2.12), (2.14) and (2.15) there are more vertical loads that contribute in the DNB load combination which gives the structure a higher total mass which in turn gives a lower natural cyclic frequency. This is presented in Table 7.1.

Table 7.1; First three natural cyclic frequency comparison.

<table>
<thead>
<tr>
<th>No</th>
<th>SMART2013-project</th>
<th>Eurocode 8</th>
<th>DNB</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.75</td>
<td>2.37</td>
<td>2.35</td>
</tr>
<tr>
<td>2</td>
<td>4.18</td>
<td>4.20</td>
<td>4.18</td>
</tr>
<tr>
<td>3</td>
<td>8.18</td>
<td>8.13</td>
<td>8.10</td>
</tr>
</tbody>
</table>

As can be seen in Table 7.1, the SMART2013-specimen shows similar values as obtained with the computerized models. The specimens natural cyclic frequencies should theoretically be the same as for the Eurocode 8 combination due to the type of loading, including structural dead load and live load. The differences between the SMART2013-specimen and Eurocode 8 can be due to vertical loads. That may act slightly different on the real structure than on the modeled structure, which is fully controlled. The natural cyclic frequencies from the SMART2013-project were obtained using a hammer shock that excited the structure to vibrate freely while the frequency was recorded using accelerometers installed on the structure.

By comparing the maximum values of internal forces from the primary seismic beam at the top level it can be seen that the difference between the maximum values can go up to 35%, which is not negligible. Although the DNB applies more vertical loads on the structure, the magnitude of the seismic action in Eurocode 8 is 40% larger than with DNB, according to Eqs. (2.12), (2.14) and (2.15). This is shown in Figures 7.1 and 7.2 and Tables 7.2 and 7.3.
Table 7.2; Beam internal force comparison of normal force $N$ and shear forces $T_y$ and $T_z$.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Eurocode 8</th>
<th>DNB</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N$</td>
<td>2000 kN</td>
<td>1800 kN</td>
<td>10</td>
</tr>
<tr>
<td>$T_y$</td>
<td>230 kN</td>
<td>150 kN</td>
<td>35</td>
</tr>
<tr>
<td>$T_z$</td>
<td>1850 kN</td>
<td>1700 kN</td>
<td>8</td>
</tr>
</tbody>
</table>

Figure 7.1; Internal force diagrams of primary seismic beam from load combinations Eurocode 8 and DNB which shows the higher obtained normal and shear forces from Eurocode 8 load combination.
Table 7.3; Beam internal force comparison of moments $M_t$, $M_y$ and $M_z$.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Eurocode 8</th>
<th>DNB</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_t$</td>
<td>350 kNm</td>
<td>265 kNm</td>
<td>24</td>
</tr>
<tr>
<td>$M_y$</td>
<td>1250 kNm</td>
<td>1125 kNm</td>
<td>10</td>
</tr>
<tr>
<td>$M_z$</td>
<td>110 kNm</td>
<td>75 kNm</td>
<td>32</td>
</tr>
</tbody>
</table>

These differences in internal forces can cause the difference between the maximum displacements in the studied primary seismic beam which goes up to 33%. See Tables 7.4 and 7.5 and corresponding Figures 7.3 and 7.4.

Figure 7.2; Internal force diagrams of primary seismic beam from load combinations Eurocode 8 and DNB which shows the higher obtained moments from Eurocode 8 load combination.
Table 7.4: Beam displacement comparison of $e_x$, $e_y$ and $e_z$.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Eurocode 8</th>
<th>DNB</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e_x$</td>
<td>4.40 mm</td>
<td>3.10 mm</td>
<td>30</td>
</tr>
<tr>
<td>$e_y$</td>
<td>6.12 mm</td>
<td>4.90 mm</td>
<td>22</td>
</tr>
<tr>
<td>$e_z$</td>
<td>3.85 mm</td>
<td>3.55 mm</td>
<td>8</td>
</tr>
</tbody>
</table>

By studying the diagrams in Figure 7.3 it can be seen that the displacement in the horizontal directions varies significantly while the displacement in the vertical direction varies less. The deflection is also presented in Figure 7.4 and Table 7.5 where the maximum difference between results obtained from the Eurocode 8 and the DNB load combination is 33% while the minimum difference between them is 8%.
Table 7.5: Beam deflection comparison of fix, fy and fz.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Eurocode 8</th>
<th>DNB</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>fix</td>
<td>0.0220°</td>
<td>0.0172°</td>
<td>22</td>
</tr>
<tr>
<td>fy</td>
<td>0.0573°</td>
<td>0.0525°</td>
<td>8</td>
</tr>
<tr>
<td>fz</td>
<td>0.0351°</td>
<td>0.0238°</td>
<td>33</td>
</tr>
</tbody>
</table>

The same calculation was made for the primary seismic column at the top level and the results are similar to those from the beam analysis. The results from the different load combinations are of a magnitude that cannot be neglected. Maximum and minimum differences in internal forces are 25% and 4.5%, respectively. The difference in displacement goes up to 32%. The results from the primary seismic column are presented in Tables 7.6 and 7.7.

Figure 7.4: Displacement diagrams of primary seismic beam from load combinations Eurocode 8 and DNB which shows the larger obtained deflections from Eurocode 8 load combination.
Table 7.6; Column internal force comparison.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Eurocode 8</th>
<th>DNB</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>2750 kN</td>
<td>2625 kN</td>
<td>4.5</td>
</tr>
<tr>
<td>$T_y$</td>
<td>270 kN</td>
<td>230 kN</td>
<td>15</td>
</tr>
<tr>
<td>$T_z$</td>
<td>210 kN</td>
<td>170 kN</td>
<td>19</td>
</tr>
<tr>
<td>$M_t$</td>
<td>43 kNm</td>
<td>32 kNm</td>
<td>25</td>
</tr>
<tr>
<td>$M_y$</td>
<td>560 kNm</td>
<td>450 kNm</td>
<td>20</td>
</tr>
<tr>
<td>$M_z$</td>
<td>640 kNm</td>
<td>560 kNm</td>
<td>12.5</td>
</tr>
</tbody>
</table>

Table 7.7; Column displacement comparison.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Eurocode 8</th>
<th>DNB</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e_x$</td>
<td>3.90 mm</td>
<td>3.60 mm</td>
<td>8</td>
</tr>
<tr>
<td>$e_y$</td>
<td>4.40 mm</td>
<td>3.13 mm</td>
<td>29</td>
</tr>
<tr>
<td>$e_z$</td>
<td>6.30 mm</td>
<td>4.70 mm</td>
<td>25</td>
</tr>
<tr>
<td>$\phi_x$</td>
<td>0.0280°</td>
<td>0.0190°</td>
<td>32</td>
</tr>
<tr>
<td>$\phi_y$</td>
<td>0.0410°</td>
<td>0.0320°</td>
<td>22</td>
</tr>
<tr>
<td>$\phi_z$</td>
<td>0.0344°</td>
<td>0.0270°</td>
<td>21</td>
</tr>
</tbody>
</table>

The overall structure response from each load combination was also calculated and illustrated in Figure 5.19 for the Eurocode 8 and in Figure 5.36 for the DNB. The structure displacement response from these load combinations are summarized in Table 7.8, which includes the maximum displacement of the structure from each combination. The differences at each point are also given which goes up to 27%. Points A-D represents four corners of the structures top slab and point E represents the maximum calculated displacement on the structure.

Table 7.8; Structure displacement comparison at each point.

<table>
<thead>
<tr>
<th>Point</th>
<th>Displacement from Eurocode 8 [mm]</th>
<th>Displacement from DNB [mm]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>9.25</td>
<td>6.78</td>
<td>27%</td>
</tr>
<tr>
<td>B</td>
<td>8.14</td>
<td>5.90</td>
<td>27%</td>
</tr>
<tr>
<td>C</td>
<td>5.21</td>
<td>3.99</td>
<td>23%</td>
</tr>
<tr>
<td>D</td>
<td>8.78</td>
<td>6.59</td>
<td>25%</td>
</tr>
<tr>
<td>E</td>
<td>11.40</td>
<td>9.52</td>
<td>16%</td>
</tr>
</tbody>
</table>
8. Conclusion

Structures such as nuclear power plants are designed to withstand earthquake and other external events. The design of such sensitive structures must be done in a way so that no external effects jeopardize the safety of the plants. Some countries e.g. France, design their nuclear power plants to withstand an earthquake twice as strong as the 1000-year event calculated for each site. This is due to the importance of the structure which should not get affected by any external loadings such as earthquakes. Due to the importance of safety and tolerability of these structures the International Atomic Energy Agency (IAEA) has provided a safety guide on seismic risks of nuclear power plants. Because of frequency and magnitude of earthquakes in Japan, design criteria’s for its nuclear facilities are far more stringent than none nuclear facilities. In Japan the power reactors are built on hard rock foundations to minimize the seismic effects, i.e. similar to ground conditions used for Swedish nuclear power plants [33].

Because of safety importance and to prevent any kinds of disasters that can relate to a nuclear facility, design criteria’s are much more reliable if it is as conservative as possible. Even though both Eurocode and DNB confirm the inapplicability of Eurocode 8 on nuclear facilities, the comparisons confirms that the Eurocode 8 gives a more conservative design criterion than what the DNB gives. This comparison is valid for normal force $N$, shear forces $T$, moments $M$, displacements $e$ and deflections $f$ in $x$, $y$ and $z$ directions i.e. every directions. The results obtained from the DNB load combination is under the influence of not including some load combination components, but these are considered neglectable compared to other components. Domination of Eurocode in the obtained results is considered due to magnification of the seismic action in its load combination, which is 40% greater than the DNB.

Earlier in this thesis, in section 3.5, Swedish design spectrum was re-scaled according to Eurocode 8 with respect to the behavior factor $q$. Due to the magnification of the difference between the re-scaled spectrum and the original Sweidish design spectrum, this reduction was not included in the analysis.

The results obtained from this thesis, which are presented in chapter 7, shows that with Swedish seismic conditions, combination of Swedish design spectrum with the load combination from Eurocode 8 gives more conservative design values than with the DNB.

For further work it should be noted that the results presented in this thesis are obtained using the response spectrum method of analysis, due to the time limit and software availability. More detailed investigations are recommended with Time Integration Method. This enables the analysis of the structure to be done during the whole time interval and not just the maximum responses.
References


